Appendix C – Batch Plant Design Criteria

HNTB Companies
The HNTB Companies
Engineers Architects Planners

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600 108th Ave. NE, Suite 900
Bellevue, WA 98104

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HNTB

Transmittal No. 5086
Date: May 5, 2011

Client Job No.
RE: Batch Plant Foundation Plans, Sections and Details-Revised Drawings

To: Ann Hegstrom
Kiewit
19472 Powder Hill Place N.E.
Poulsbo, WA 98370
(253) 200-3507

Transmitting:

- ☐ Estimates/Proposals
- ☐ Change Order
- Minutes
- ☐ Plans/Specifications
- ☐ Shop Drawings
-] Other
- ☐ Correspondence
- ☒ Submittal
- ☐ Shop Drawings
- ☐ Technical Data

By Way Of: Centric

No. Copies	Drawing Number	Date/Rev	Description
Electronic	Rev 2 Sr 520 BP fnd.pdf	05/05/2011	Batch Plant Foundation Plans, Sections and Details (2 Page)

Transmitted For:

- ☐ Approval
- ☐ As Requested
- ☐ Distribution/Construction
- ☐ Your Use
- ☒ Submit
- ☐ Correct as Noted/Resubmit
- ☐ Return
- ☐ Amend/Resubmit
- ☐ Review and Comment

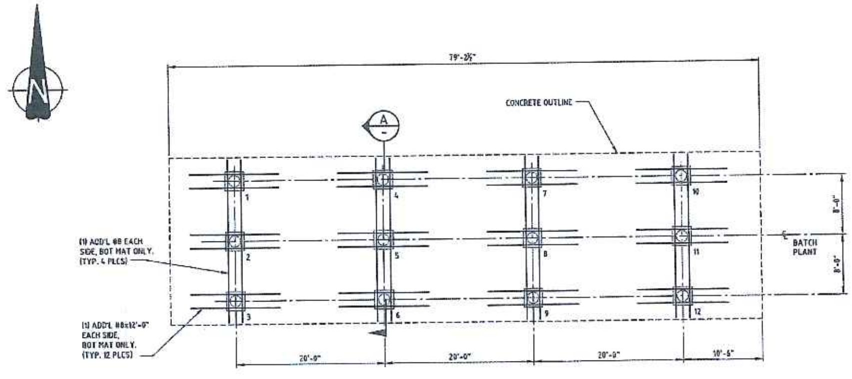
Copied To: File; Tom Schnetzer, Jim Peterson

Sent By: Arbert Harris

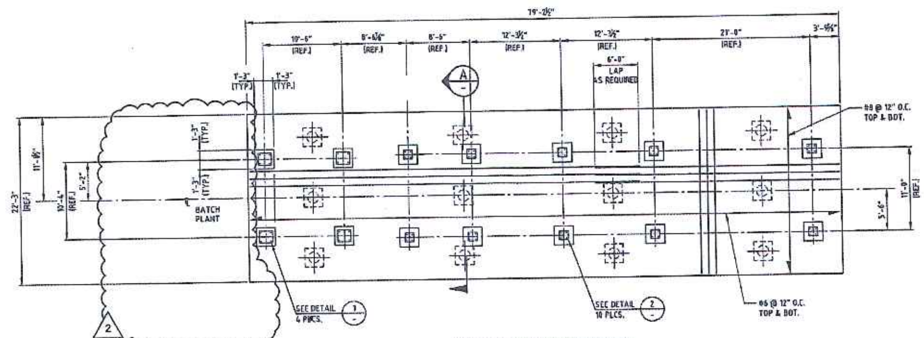
Received By: ____________________

Date: May 5, 2011

Date: ____________________



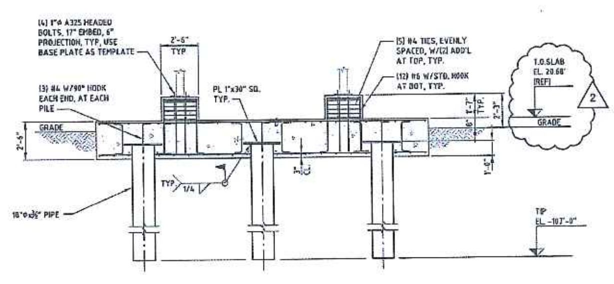
BATCH PLANT PILE PLAN
1/8"=1'-0"



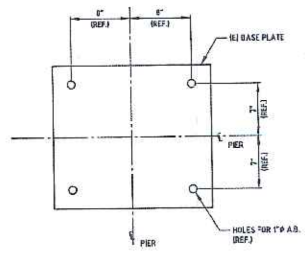
BATCH PLANT CONCRETE PLAN
1/8"=1'-0"

NOTES:

1. FOUNDATION DESIGN WIND: 105 MPH EXP. C.
2. CONCRETE f'_c=4000 PSI.
3. PILE MATERIAL ASTM 252 GR 35.
4. FIELD VERIFY ALL REFERENCE (REF.) DIMENSIONS.
5. REINFORCING STEEL ASTM A615 GR 60.
6. (E) EXISTING.
7. WELD USING E70XX ELECTRODES BY QUALIFIED WELDERS, USING AWS D1.1 PROCEDURES.
8. PLATE ASTM A36.
9. STEEL FABRICATION PER AISC 13TH EDITION.
10. ALL CONCRETE SHALL CONFORM TO ACI 318-05. ¾" CHAMFER ALL EXPOSED EDGES.
11. ALL CONSTRUCTION SHALL CONFORM TO IBC 2009 WITH WASHINGTON AMENDMENTS AND WASHINGTON HEALTH AND SAFETY ADMINISTRATION.
12. DRIVING RECORDS/BLOW COUNTS SHALL BE MAINTAINED FOR THE FINAL 30' TO 40' OF THE INSTALLATION OF THE TOP 65' SECTION OF PIPE.
13. PILE LOCATION TOLERANCES:
    MAXIMUM VARIATION FROM CUT-OFF ELEVATION = 2"
    MAXIMUM OUT OF POSITION = 3"
    ALIGNMENT: 1/2" HORIZONTALLY IN 2 FEET VERITICALLY FROM PLUMB.
14. THE BOTTOM 65' OF THE PILE WILL BE FABRICATED FROM SALVAGED PIECES OF PIPE OF VARYING LENGTH, BUT NO ONE PIECE WILL BE LESS THAN 10' LONG. EACH PIECE WILL HAVE ONE END BEVELLED. THE PIECES WILL BE SHOP WELDED TOGETHER USING FULL PENETRATION SINGLE VEE WELDS WITH AT LEAST A 1/8" CONTINUOUS RING BACKER BAR. THE SHOP FABRICATED CLOSED END 65' BOTTOM PIECE WILL BE SHIPPED TO THE FIELD AND DRIVEN TO JUST ABOVE GRADE. A SINGLE PIECE OF PIPE VARYING IN LENGTH FROM 65' TO 70', AS REQUIRED, WILL BE FIELD WELDED TO THE ALREADY DRIVEN PIPE. THE FIELD WELD WILL BE A SINGLE VEE FULL PENETRATION WELD WITH 0.25"x2" CONTINUOUS BACKER RING. ALL WELDERS WILL BE WABO CERTIFIED AND QUALIFIED FOR THE WELD PROCEDURES. ALL SHOP AND FIELD WELDS WILL BE CONTINUOUSLY INSPECTED AND FULLY TESTED IN ACCORDANCE WITH THE IBC AND AWS D1.1 BY WABO CERTIFIED INSPECTORS.

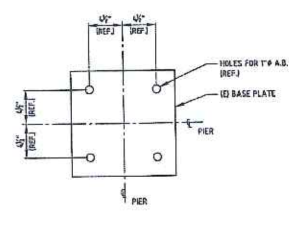


SECTION A
1/4"=1'-0"



(SUPPLIED BY OTHERS)

DETAIL 1
1 1/2"=1'-0"



(SUPPLIED BY OTHERS)

DETAIL 2
1 1/2"=1'-0"

REV	DATE	REVISION DESCRIPTION	DRAWN	DRAFT CHK	DESIGNED	ENG CHK	APPROVED	CUSTOMER	REF DRAWING No	REFERENCE DRAWING TITLE
2	25-APR-11	REVISE ELEVATION CALLOUT & REMOVE APRON	DAS		RRL		JLC	CALPOR		
1	20-APR-11	REVISED PEDESTALS & ADDED NOTE	DAS		RRL		JLC	CALPOR		
0	25-MAR-11	ISSUED FOR CONSTRUCTION	DAS	HER	RRL	CAG	JLC	CALPOR		

D SHEET | SCALE SHOWN

OneWay

WORLEYPARSONS PROJECT No 80547

ENGINEERING AND PERMIT STAMPS (As Required)

JENNIFER L. CARLSON · STATE OF WASHINGTON · PROFESSIONAL ENGINEER

CUSTOMER

PORTLAND

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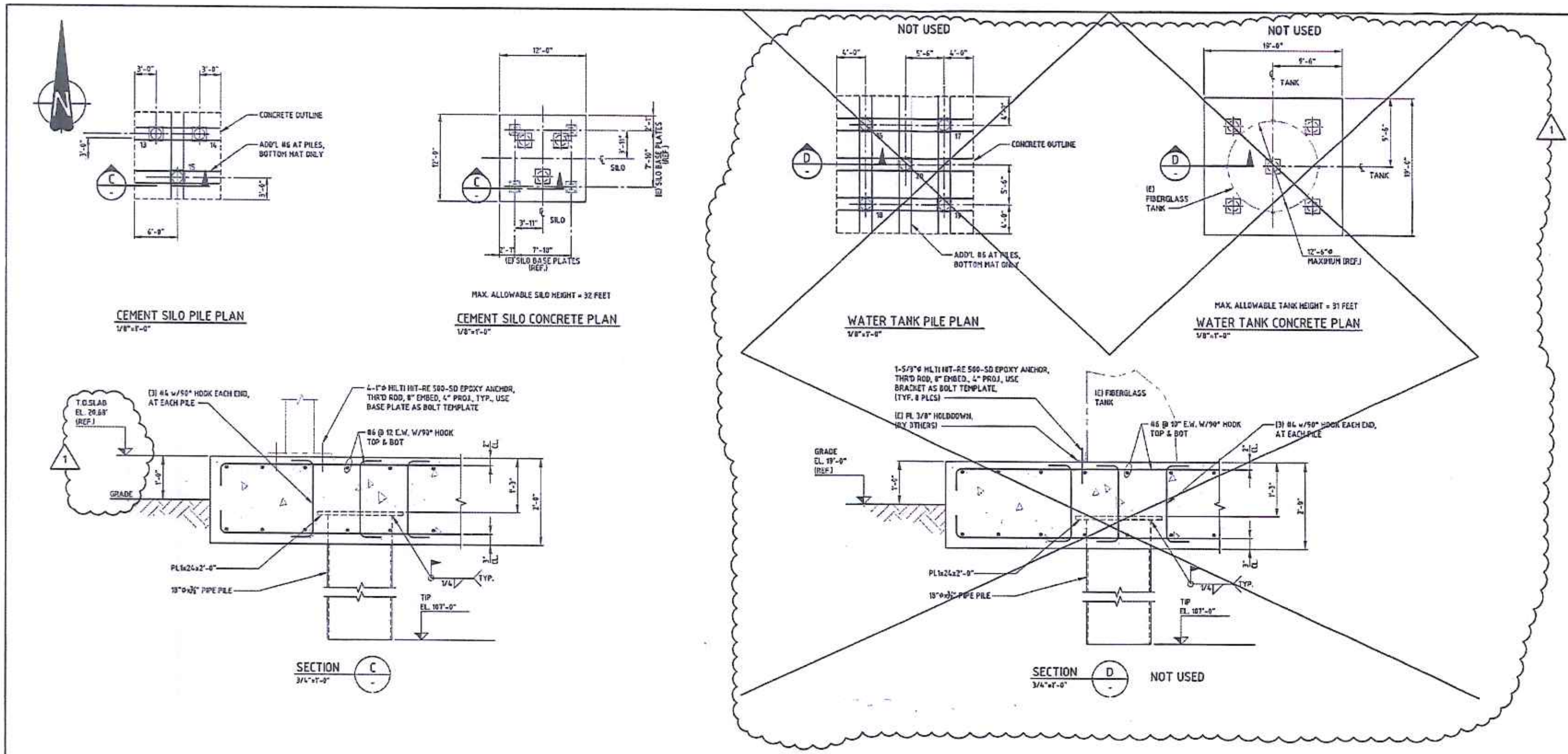
resources & energy

SR520 PONTOON BRIDGE
PONTOON BATCH PLANT
FOUNDATION PLANS, SECTIONS & DETAILS

DWG No 80547-ST-DGA-2001 | REV 2

PLOT DATE & TIME: 25/4/2011 4:26:34 PM
SAVE DATE & TIME: 25/4/2011 4:26:31 PM
USER NAME: david.sund
LOCATION: X:\2010\PORTLAND PROJECT WORK\80547 - 520 PONTOON BRIDGE BATCH PLANT FOUNDATION DESIGN\DRAFTING\STRUCT\80547-ST-DGA-2001.DWG



NOTES:

1. FOR GENERAL NOTES SEE DGA-2001.

REV	DATE	REVISION DESCRIPTION	DRAWN	DRAFT CHK	DESIGNED	ENG CHK	APPROVED	CUSTOMER	REF DRAWING No	REFERENCE DRAWING TITLE
1	25-APR-11	REVISE ELEVATION CALLOUT & ADD "NOT USED" NOTES	DAS		RRL		JLC	CALPOR		
0	25-MAR-11	ISSUED FOR CONSTRUCTION	DAS	MER	RRL	CAG	JLC	CALPOR		

D SHEET SCALE SHOWN

OneWay

WORLEYPARSONS PROJECT No 80547

ENGINEERING AND PERMIT STAMPS (As Required)

JENNIFER L. CARLSON — STATE OF WASHINGTON — PROFESSIONAL ENGINEER

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SR520 PONTOON BRIDGE
PONTOON BATCH PLANT
FOUNDATION PLANS AND SECTIONS

DWG No 80547-ST-DGA-2002 REV 1

CALCULATIONS
TABLE OF CONTENTS

520 BRIDGE CASTING PLANT
BATCH PLANT, TREATMENT TANK, AND SILO FOOTINGS

January, 2011
Victor Ghiglieri, P.E. (WA)
Alice Maupin, P.E. (CA)

VICTOR J. GHIGLIERI
STATE OF WASHINGTON
37366
REGISTERED
PROFESSIONAL ENGINEER
1/14/10
EXPIRES 06/26/11

REGISTERED PROFESSIONAL ENGINEER
ALICE E. MAUPIN
NO. 51691
Exp. 6/30/12
CIVIL
STATE OF CALIFORNIA

SR520 BRIDGE BATCH PLANT
PILE FOUNDATION DESIGN
January, 2011
By: AM

Required: Design slab and piles for portable batch plant.

Location: Aberdeen, WA

References:

1. GeoTechnical Report by Shannon & Wilson, attached
2. Con-E-Co drawing C7362-F1, including dead, live, and wind loads

2a. Con-E-Co drawing C7362-F1 calculation markup

3. ACI 318-05, "Building Code Requirements for Structural Concrete"
4. "Design of Headed Anchor Bolts" by Shipp & Haninger, att.

Given:

$f'_c := 4000 \cdot psi$ — concrete compressive strength

$F_y := 60000 \cdot psi$ — ASTM A615, Grade 60 reinforcing bars

Overturning

Ref. 2a

$$M_r := \left[\left[(81)\cdot\left(\frac{22.25}{2}\right)\cdot(2.0)\right]\cdot ft^3 \cdot 150 \cdot \frac{lb}{ft^3}\right]\cdot\left(\frac{22.25\cdot ft}{2}\right) + (32.7\cdot k)\cdot(5.625\cdot ft + 16.625\cdot ft) \qquad M_r = 3735\cdot k\cdot ft$$

$M_{OT} := 825\cdot k\cdot ft$

$$\frac{M_r}{M_{OT}} = 4.53 \quad > \quad 1.0 \qquad \text{OK}$$

$\boxed{\text{No uplift}}$

Soil Pressure

Ref. 2a

$p := \frac{(134.1 + 134.1 + 25)\cdot k}{200.25\cdot ft^2}$ $\qquad p = 1.46\cdot ksf$

Punching Shear

$d := 24\cdot in - \left(3 + \frac{.625}{2}\right)\cdot in$ $\qquad \phi := 0.75$ $\qquad d = 20.69\cdot in$

$b_o := 4\cdot(14\cdot in + d)$ $\qquad b_o = 138.7\cdot in$

$V_c := 4\cdot\sqrt{f'_c\cdot psi}\cdot b_o\cdot d$ $\qquad \phi\cdot V_c = 545\cdot k \; > \; 1.6\cdot(134.1\cdot k) = 215\cdot k$ Ref. 3, 11-35

OK

Beam Shear

See attached diagram

$w := 1500\cdot plf$ $\qquad L := 5.01\cdot ft$ $\qquad M_u := \frac{w\cdot L^2}{2}$ $\qquad M_u = 18.83\cdot k\cdot ft$

$b := 12\cdot in$ $\qquad F := \frac{b\cdot d^2}{12000}\cdot\frac{1}{in^3}$ $\qquad F = 0.428$ $\qquad \phi := 0.9$

$K_n := \frac{M_u}{\phi\cdot F\cdot(k\cdot ft)}$ $\qquad K_n = 49$ $\qquad$ Use minimum bending steel

$\rho_{min} := 1.33\cdot 0.0013$ $\qquad \rho_{min} = 0.0017$ $\qquad \rho_{min}\cdot b\cdot d = 0.43\cdot in^2$ $\qquad$ Ref. 3, 10.5.3

Use #6 @ 12" o.c. each way at top and bottom.

3

Development Length

$d_b := .75 \cdot in$

$L_d := \frac{F_y}{25 \sqrt{f'_c \cdot psi}} \cdot d_b$ $\qquad L_d = 28.5 \cdot in$ $\qquad$ Ref. 3, 12.2.2

18" Diam. x 3/8" Steel Piles

$w := 1500 \cdot psf$

$w \cdot (8.25 \cdot ft) \cdot s = (0.65) \cdot 400 \cdot k$ solve, $s \rightarrow 21.0101010101010101 \cdot ft$ $\qquad$ Ref. 1, Fig. 2

Head Plate Punching Shear

$d := 12 \cdot in$ $\qquad$ depth of head plate

$b_o := 4 \cdot (d + 30 \cdot in)$ $\qquad b_o = 14 ft$

$V_c := 4 \cdot \sqrt{f'_c \cdot psi} \cdot b_o \cdot d$ $\qquad V_c = 510 \cdot k$

$\phi := 0.75$

$\phi \cdot V_c = 383 \cdot k$ $\quad > \quad$ pile capacity

OK

Headed Anchor Bolts

See Ref. 4.

$T_F := 31.6 \cdot k$ $\qquad V_l := 12.8 \cdot k$ $\qquad$ Maximum wind uplift and horizontal loading, columns A1 & A2

$\phi := 0.55$

$C := 1.85$ $\qquad$ For grouted baseplate

$\alpha := 1.0$

$T := \left(\frac{C \cdot V_l + T_F}{\phi} \right)$ $\qquad T = 100.5k$

$F_y := 92 \cdot ksi$ $\qquad F_u := 120 \cdot ksi$ $\qquad$ A325 bolt

$d := 1.0 \cdot in$

$A_t := 0.606 \cdot in^2$ Tbl. 2A, 1" diam. bolt

$n := 4$ 4 bolts per column

$n F_y \cdot A_t = 223\,k$ $> T$

$A_{col\_reqd} := \dfrac{72720 \cdot lb}{4(0.65) \cdot \sqrt{f_c \cdot psi}}$ $A_{col\_reqd} = 442\,in^2$ $<$ $A_{col} := (30 \cdot in)^2$ OK

<u>Vertical reinforcement</u>

$F_y := 60 \cdot ksi$ For rebar

$A_{st\_reqd} := \dfrac{n \cdot (72720 \cdot lb)}{F_y}$ $A_{st\_reqd} = 4.85\,in^2$

$A_{st} := 12 \cdot \left(0.44 \cdot in^2\right)$ $A_{st} = 5.28\,in^2$ OK

$\boxed{\text{Use 12 - \#6 bent bars}}$

$L_{d\_reqd} := 17 \cdot d$ $L_{d\_reqd} = 17\,in$

<u>Ties</u>

$A_{sv} := \dfrac{F_u \cdot A_t}{C \cdot F_y \cdot \cos\left(\frac{\pi}{4}\right)}$ $A_{sv} = 0.93\,in^2$

$\dfrac{A_{sv}}{0.196 \cdot in^2} = 4.73$

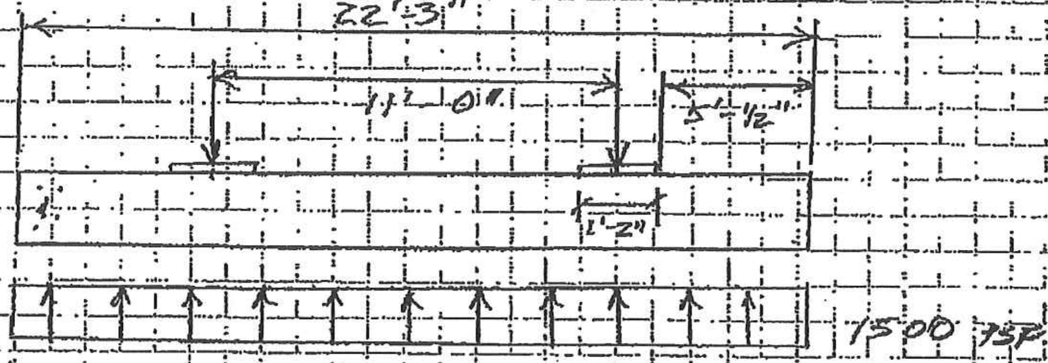
$\boxed{\text{Use 5 - \#4 ties @ 3" o.c.}}$

<u>Column shear</u>

$\phi := 0.75$

$V_c := 2 \cdot \sqrt{f_c \cdot psi} \cdot \left(30 - 3.0 - 0.5 - \frac{.75}{2}\right) \cdot in \cdot (30 \cdot in)$ $\phi \cdot V_c = 74.4\,k$ $>$ V_i OK

$\boxed{\text{Use 30"x30" concrete column}}$


22'-3"
11'-0"
5'-1/2"
1'
1'-2"
1500 PSF

6

NOTES:

1) k = 1000 LBS.

2) ALL LOADS OCCUR AT THE BOTTOM OF BASE PLATES, TOP OF FOUNDATION.

3) WIND LOADS ARE BASED ON UNIFORM BUILDING CODE (1997), 90 MPH

4) CONSULT WITH A REPUTABLE PROFESSIONAL ENGINEER FOR THE DESIGN OF THE FOUNDATION.

5) THE CONCRETE EQUIPMENT CO. DOES NOT ASSUME RESPONSIBILITY FOR FOUNDATION DESIGN.

6) EXTEND FOOTINGS BELOW FROST LINE (MIN. DEPTH – 4'-0")

7) MINIMUM WELD PLATE SIZE FOR MIXER BEARING SHOULD BE 24" X 28".

8) TO CALCULATE THE BASE PLATE BEARING STRESSES IT IS ASSUMED THAT THE MINIMUM COMPRESSIVE STRENGTH OF THE CONCRETE IS 3000 PSI.

9) THIS PLANT IS NOT DESIGNED FOR EARTHQUAKE LOADS

INDIVIDUAL LOADS — FOUNDATION LOADS SUMMARY
UNBALANCED WITH 400 BBL CEMENT EXTENSION

COLUMN	DEAD "D"	AND LIVE "L"	VERT WIND "W"	DOWNLOAD SUMMARY D+L+W	UPLOAD WIND W-D	HORIZ WIND	WIND OVERTURN MOMENT KIP-FT
F1	9.9	55.7	35.8	101.4	26.0	14.2	266.3
F2	9.9	55.7	35.8	101.4	26.0	14.2	266.3
A1	6.6	88.9	38.4	134.1	31.8	12.8	228.6
A2	6.6	88.9	38.4	134.1	31.8	12.8	228.6
B1	4.5	83.1	10.3	97.9	5.8	4.2	113.5
B2	4.5	83.1	10.3	97.9	5.8	4.2	113.5
C1	6.5	86.6	10.1	103.1	3.6	6.2	110.7
C2	6.5	86.6	10.1	103.1	3.6	6.2	110.7
E1	5.0	70.0	10.1	85.1	5.1	5.5	111.1
E2	5.0	70.0	10.1	85.1	5.1	5.5	111.1

MIXER FOUNDATION LOADS
(MIXER WITH TRANSPORT CARRIAGE, BAG HOUSE, CANTILEVERED MUD HOPPER)
MIXER LOADS PER COLUMN DEPEND ON THE TILTING POSITION OF THE MIXER DRUM

	DEAD D	MAX. LIVE L	+ OR - VERT. WIND W	COMB. VERT. D+L+W	LIVE LOAD VERT. UP OR DOWN W-D-L	*VERT. IMPULSE	HORIZ. WIND	*HORIZ. IMPULSE
J1	4.9	34.3	22.1	61.9	17.0	19.5	7.3	4.0
J2	4.9	34.3	22.1	61.9	17.0	19.5	7.3	4.0
K1	15.2	30.8	22.1	68.1	2.9	3.8	7.3	4.0
K2	15.2	30.8	22.1	68.1	2.9	3.8	7.3	4.0
F1					-3.7	-4.5		4.0
F2					-3.7	-4.5		4.0
A1					-10.2	-12.6		4.0
A2					-10.2	-12.6		4.0

NEGATIVE NUMBERS IN THIS CHART ARE UPLIFT LOADS

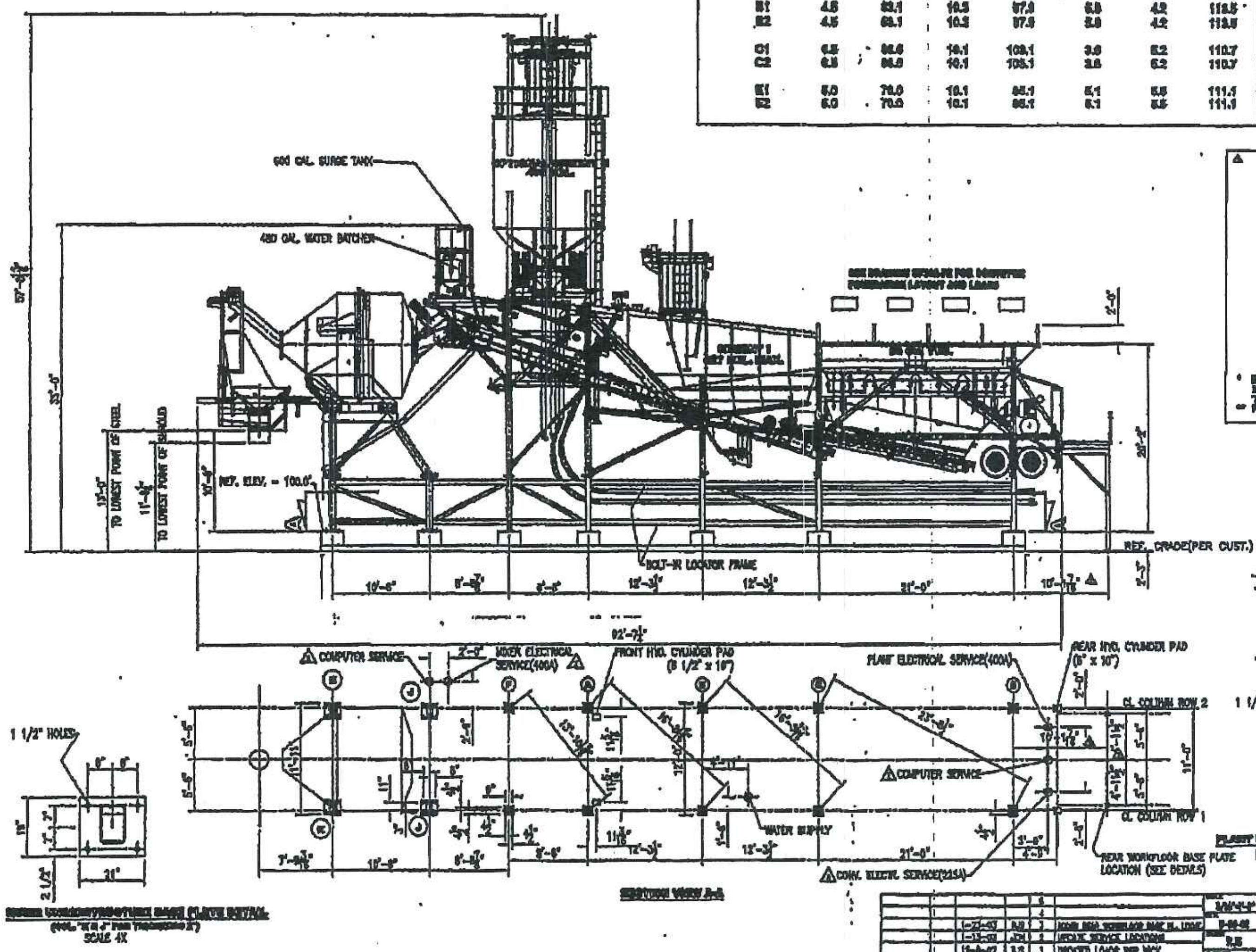
* IMPULSE LOAD IS THE FOUNDATION LOAD RESULTING FROM THE SUDDEN ACCELERATION OF THE MIXER AND TILT FRAME. FOUNDATION LOAD FROM THIS FORCE IS OF SHORT DURATION (LESS THAN ONE SECOND).

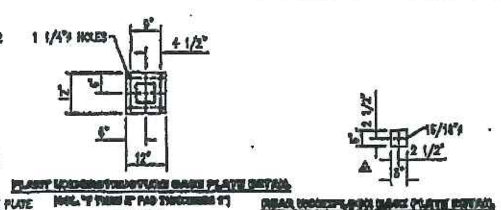
** FOUNDATION CONNECTIONS MUST RESTRAIN THESE LOADS.

MIXER TILTING AND JERKING LOAD EFFECTS

COLUMN	LIVE MOMENT VERTICAL PER COLUMN	IMPULSE MOMENT VERTICAL PER COLUMN
J	11.0	13.5
K	2.9	3.5
F	-3.7	-4.5
A	-10.2	-12.6

NEGATIVE NUMBERS IN THIS CHART ARE UPLIFT LOADS





PLANT SUBSTRUCTURE BASE PLATE DETAIL

SCALE 4X

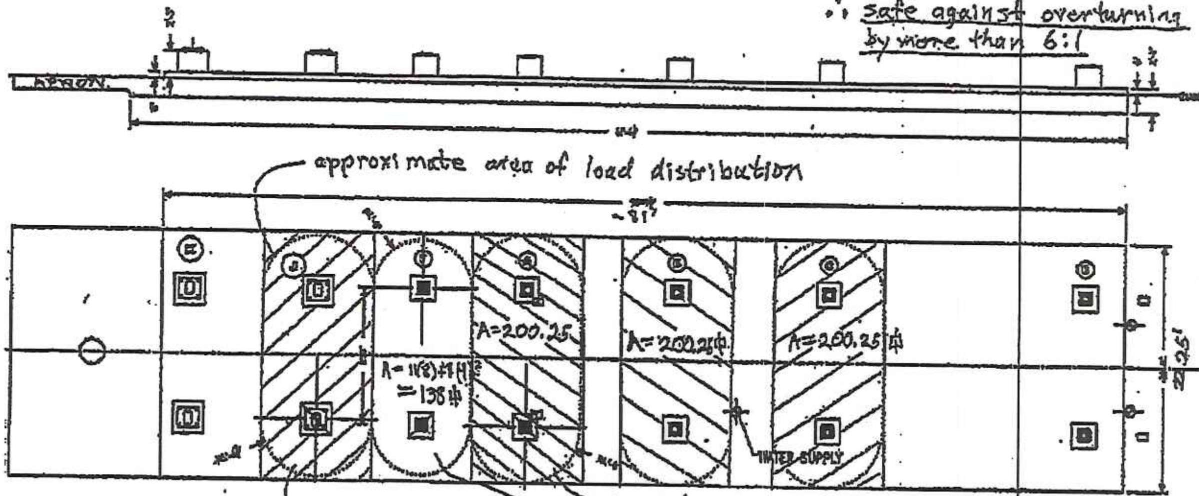
CON-E-CO
concrete equipment company

LAYOUT & MIXER FOUNDATION LAYOUT

Note: All loads are from Con-E-Co dwg. C7362-F1
See dwg. notes

W slab → $22.25' \times 81' \times 2' \times 150\frac{\#}{ft^3} = 541\ k$

$\frac{3604.5 ft^3}{27 ft^3/yd} = 133.5\ yd^3$ + columns & apron



→ $A = 9(22.25)$
$= 200.25\ ft$
Typical columns J,A,B,C

→ worst case loading for A = 200.25 ft² (columns B)

Column Loads from dwg. C7362-F1 D+L+W → $\frac{134.1 + 134.1 + 25}{200.25} = \frac{293.2}{200.25} = 1.47\ k/ft^2$

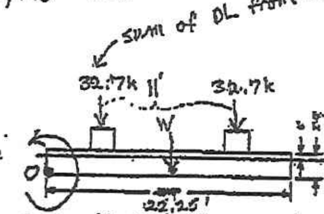
→ worst case loading for A = 138 ft² (columns F)

→ $\frac{101.4 + 101.4}{138} = 1.47\ k/ft^2 < 1.5\ k/ft^2$, ∴ O.K. for 1,500 psf soil bearing capacity

✓ Overturning From Wind Load

$\Sigma M_o = 32.7(5.625) + 32.7(16.625) + 541(10) - 825\ k\text{-}ft$
$= 183.94 + 543.64 + 5410 - 825$
$= 5,312.6\ k\text{-}ft$

∴ safe against overturning by more than 6:1



825 k-ft (sum of moments from dwg. C7362-F1) (Wind Loads)

Foundation Weight on soil buried 18" for 200 ft section is:
$200(150-100)1.5' + 200(150).5' = 25k$
(for soil weight of 100 pcf, concrete @ 150 psf)

Note: Foundation loading assumes 90 mph wind blowing from both sides of structure at once. Actual loading would of course be l...

8

Design of Headed Anchor Bolts

JOHN G. SHIPP AND EDWARD R. HANINGER

In current practice the design of base plates is controlled by bearing restrictions on the concrete (see Fig. 1); shear is transmitted to the concrete largely through anchor bolts, shear lugs or bars attached to the base plate and the tensile anchorage steel is generally proportioned only for direct stress. The embedment requirements for anchorage steel are not clearly defined by most codes and are left largely to the discretion of the design engineer. Also, there are no provisions to prevent a brittle failure in the concrete as opposed to a ductile failure in the anchor bolt, as provided for with a probability-based limit states design or Load and Resistance Factor Design (LRFD) for steel.[8] Larger design forces now mandated in many areas due to the revised seismic and wind loads require design capacities for anchor bolts beyond any existing code values.[6,11] Therefore, there is a need for a complete design procedure for anchor bolts that will accommodate these larger loads and incorporate the proposed design philosophy, i.e., probability-based limit states design (PBLSD).[8]

THE HEADED BOLT AS AN ANCHORAGE

The *headed bolt*, as designed herein, is recommended as the most efficient type of anchorage to use for both tension and shear loads. Other anchorages which have been used are L-bolts, J-bolts, rods with a bolted bearing plate and shear lugs. L-bolts have been shown to be less effective in resisting slip at service load levels than headed bolts.[13] The authors are not aware of any published data that addresses the performance of J-bolts. For a threaded rod with a bolted washer or bearing plate embedded in concrete, tests have shown that unless the plate is properly sized it may actually decrease the anchor capacity by causing a weakened failure plane in the concrete.[7,17] Shear lugs can fail in a brittle mode if not properly confined, and do not lend themselves to a shear friction analysis.[7,17]

The headed bolt, when properly embedded and confined, will develop the full tensile capacity of even A490 high strength bolts.[3] When the tension capacity of the bolt is developed, a ductile failure can be ensured by the shear friction mechanism.[3]

In this paper, anchor bolt design ductility is assured by causing a failure mechanism that is controlled by yielding of the anchor bolt steel, rather than brittle tensile failure of concrete. This is accomplished by designing the pullout strength of the "concrete failure cone" (U_c) such that it equals the minimum specified tensile strength ($F_u A_t$) or "full anchorage value" of the anchor bolt. See Figs. 2 and 10 for illustrations of the concrete failure cone concept. See Appendix A for the derivation of L_d to satisfy this criteria. The design approach presented herein is compatible with the proposed AISC Specification for Nuclear Facilities,[5] ACI 318-77,[2] and the proposed revisions to ACI 318-77.[7] The governing design approach is that presented in ACI 349, Supplement 1979.[3]

DESIGN PARAMETERS

The design approach presented is generally applicable to any of a number of bolt or concrete strengths. However, the following representative materials are used in developing the design values. Anchor bolt materials used are ASTM A36, A307 (Grade B), A325, A449 and A687. Concrete is assumed to have a minimum compressive strength (f'_c) of 3,000 psi. Anchor bolts are heavy hex bolts or threaded steel bars with one heavy hex nut placed in concrete. Bolt threads at the embedded end of each threaded steel bar are "staked" at two places below the heavy hex nut. All bolts are brought to a "snug tight" condition as defined by AISC[4] to ensure good contact between attachments. The concrete is at least 14 days old prior to tightening the anchor bolts in order to prevent bolt rotation. Anchor bolts are designed for combined shear and tension loads; the area of steel required for tension and shear is considered additive. Criteria will be presented such that either Working Stress Design (WSD) or Ultimate Strength Design (USD) may be used.

COMBINED TENSION AND SHEAR

Many authors have presented data and interaction equations to account for the combined effects of tension and shear

John G. Shipp is Supervising Structural Engineer, Fluor Engineers and Constructors, Inc., Irvine, California.

Edward R. Haninger is Senior Structural Engineer, Fluor Engineers and Constructors, Inc., Irvine, California.

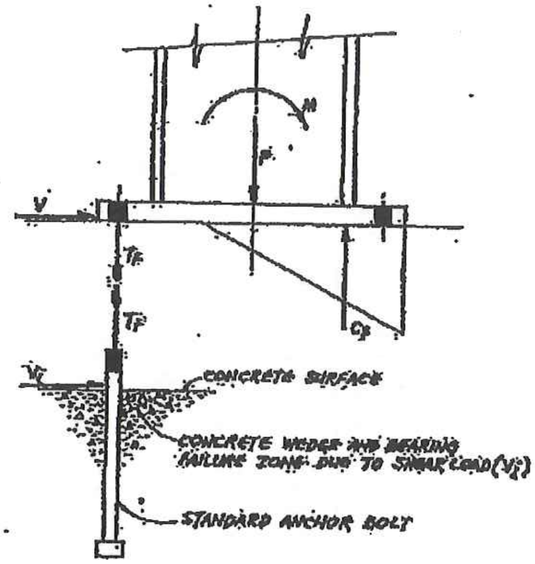


Fig. 1. Example of base plate loading

(see Refs. 1, 3, 12, 14, 15 and 17). In this paper, the total required area of anchor bolt steel to resist tension and shear loads is considered to be additive (see Appendix B, and Figs. 1 and 9).

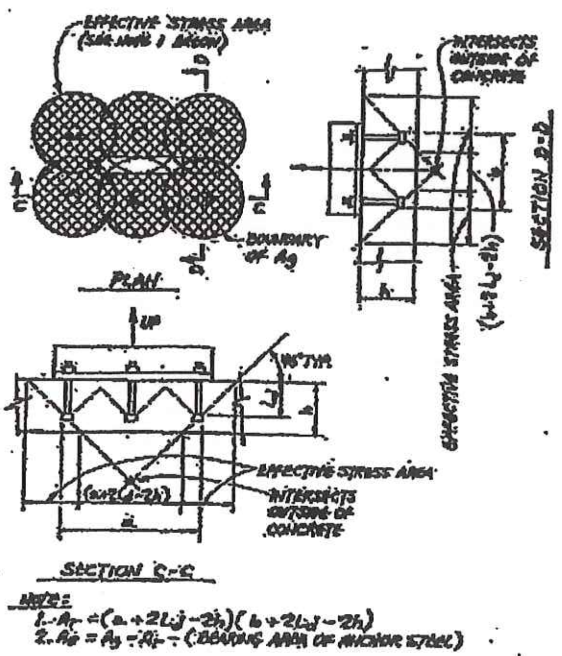


Fig. 2. Effective stress area for limited depth (A_e)

Table 1A. Standard Anchor Bolt Basic Types

Type	Description	Bolt Spacing r	Edge Distance m	Comments
A	Isolated	$r \geq r_m$	$m \geq m_v$	$m_v > r_m/2$, $m_v > m_t$
B	Shear reinforcement only	$r \geq r_m$	$r_m/2 < m < m_v$	$r_m/2 > m_t$
C	Shear reinforcement plus overlapping failure cones	$r < r_m$	$m_t < m < m_v$	$m_t < r_m/2$
D	Tension lap w/ reinforcement	$r < r_m$	$m_t < m < r_m/2$	concrete pier

Note: The bolt embedment depth shall be greater than or equal to L_d, as given in Table 1B for all bolt types.

The rationale for this basis is that the shear force (V_I) causes a bearing failure near the concrete surface and translates the shear load on the anchor bolt into an effective tension load by shear friction. In the absence of tension load (T_F), an anchor bolt is developed for "full anchorage" to resist shear. In terms of Probability-Based Limit States Design (PBLSD), the anchor bolt design resistance is greater than or equal to the effective combined tension (T_F) and shear (V_I) load effects as indicated below (see Appendix C).

$$A_t F_y \geq T$$

where

$A_t F_y$ = Nominal design resistance (capacity) equal to the product of the bolt tensile area (A_t) and the minimum specified steel yield strength (see Table 2A

$$T = \left[\frac{CV_I + T_F}{\phi}\right]\alpha$$

C = Shear coefficient, equal to the inverse of the shear friction value, as per Ref. 3, for the particular base plate mounting.

Table 1B: Values for L_d, r_m, m_v, m_t

Bolt Type (ASTM)	Development Length L_d	Minimum Bolt Spacing r_m	Minimum Edge Distance for Shear m_v	Minimum Edge Distance for Tension m_t
A307	12d	16d	12d	5d or 4" min.
A325	17d	24d	17d	7d or 4" min.
A449	17d	24d	17d	7d or 4" min.

Note: The above values were derived per Table 2B and tabulated in Table 2A for various bolt diameters.

Table 2A. Standard Anchor Bolt Basic Design Values

Bolt Diameter d (in.)	Tensile Stress Area A_t (in.²)	A_tF_y (kips) $F_y = 36$ ksi A36 A307	$F_y = 58$ ksi A449	$F_y = 81$ ksi A325 A449	$F_y = 92$ ksi A325 A449	$F_y = 105$ ksi A687	L_d; m_v (in.) 12d A36 A307	17d A325 A449	19d A687	m_t (in.) 5d or 4" min. A36 A307	7d or 4" min. A325 A449 A687	r_{bc} (in.) 16d A36 A307	24d A325 A449	28d A687
½	0.142	5.12			13.06	14.91	6	8½	9½	4	4	8	12	14
⅝	0.226	8.14			20.79	23.73	7½	11	12	4	4⅜	10	15	18
¾	0.334	12.02			30.73	35.07	9	13	14½	4	5¼	12	18	21
⅞	0.462	16.64			42.40	48.51	10½	15	17	4⅜	6⅛	14	21	25
1	0.606	21.82			55.75	63.63	12	17	19	5	7	16	24	28
1⅛	0.763	27.46		61.80		80.12	13½	19	21½	5⅝	7⅞	18	27	32
1¼	0.969	34.89		78.49		101.7	15	21¼	24	6¼	8¾	20	30	35
1⅜	1.155	41.59		93.56		121.3	16½	24	26	6⅞	9⅝	22	33	39
1½	1.405	50.59		113.80		147.5	18	25½	28½	7½	10½	24	36	42
1¾	1.90	68.4	110.2			199.5	21	30	33½	8¾	12¼	28	42	49
2	2.50	90.0	145.0			262.5	24	34	38	10	14	32	48	56
2¼	3.25	117.1	188.5			341.3	27	39	43	11¼	15¾	36	54	63
2½	4.00	144.0	232.0			420.0	30	43	48	12½	17½	40	60	70
2¾	4.93	177.5	285.9			517.7	33	47	52	13¾	19¼	44	66	77
3	5.97	214.9	346.3			626.9	36	51	57	15	21	48	72	84

Notes:

1. The following formulas have been conservatively simplified by using the values in Table 2B:

   (a) $L_d = 12d\sqrt{\dfrac{F_u}{58}}$ per ACI-349 Appendix B, Sect. B.4.2

   (b) $m_t = d\sqrt{\dfrac{F_u}{56\sqrt{f'_c}}}$ per ACI-349 Appendix B, Sect. B.5.1.3

   (c) $m_v = d\sqrt{\dfrac{F_u}{7.5\sqrt{f'_c}}}$ per ACI-349 Appendix B, Sect. B.5.1.1

2. Before entering this table, the total effective design load (T) shall include the appropriate load factors, stress increase factors or probability factors, capacity reduction factors (ϕ) and shear coefficient (C).
3. All computations are based on $f'_c = 3000$ psi.

For PBLSD or Ultimate Strength Design (USD):

V_F = Shear design load effect equal to the product of the load factor(s) and the nominal shear load. The load factors are in accordance with applicable codes. For example, using ACI 318-77, $V_F = 1.4D + 1.7L$

T_F = Tension design load effect equal to the product of the load factor(s) and the nominal tension load. The load factors shall be in accordance with applicable codes. For example, using ACI 318-77, $T_F = 1.4D + 1.7L$

ϕ = Capacity reduction factor
= 0.90 for factored design loads under USD

Table 2B. Values Based on ACI 349-76 Provisions

F_u (ksi)	$L_d = 12d\sqrt{\dfrac{F_u}{58000}}$ Actual	Use	$m_t = d\sqrt{\dfrac{F_u}{56\sqrt{f_c}}}$ Actual	Use	$m_v = d\sqrt{\dfrac{F_u}{7.5\sqrt{f_c}}}$ Actual	Use
58	12d	12d	4.34d	5d	11.88d	12d
90	14.95d	17d	5.42d	7d	14.80d	17d
105	16.15d	17d	5.85d	7d	15.99d	17d
120	17.26d	17d	6.25d	7d	17.09d	17d
150	19.30d	19d	6.99d	7d	19.10d	19d

Note: Values listed in this table are based on $f'_c = 3000$ psi.

$\alpha = 1.0$ for USD. Probability considerations are included in the load factors.

For Working Stress Design (WSD):

V_i = Nominal shear load. For example, $V_i = D + L$

T_f = Nominal tension load. For example, $T_f = D + L$

ϕ = Capacity reduction factor, which includes a safety factor, used to convert yield capacity to working loads = 0.55

α = Probability factor (PF) or reciprocal of the stress increase factor ($1/SIF$), i.e., seismic loads combined with dead loads and live loads. $PF = 0.75$; therefore, $\alpha = PF = 0.75$. $SIF = 1.33$; therefore, $\alpha = 1/SIF = 0.75$.

ANCHOR BOLT DESIGN

The following section establishes limitations for the combined effects of bolt spacing, embedment depth and edge distance, such that the heavy hex head on a standard anchor bolt provides "full anchorage" in concrete equal to the tensile capacity of the bolt. Several agencies/authors have published reports representing their test data and/or recommendations to account for these variables, (see Refs. 9, 10, 13, 16 and 17). The recommendations which follow represent a composite of the published literature, modified for compatibility with ACI 349.[3] Where plain bars are used, the equivalent anchorage may be accomplished by threading the embedded end of the bar and using one American Standard heavy hex nut of equal or higher strength steel with bolt threads "staked" at two places below the heavy hex nut.

Refer to Tables 1A and 1B for a summary of the various anchor bolt classifications and criteria for which design procedures are herein provided. Note that anchor bolts are defined as type A, B, C or D. These types represent various design conditions of anchor bolts such as spacing, edge distance and development length.

Type A Anchor Bolts—Anchor bolts are classified as Type A, or isolated, when all the following apply:

- The closest bolt spacing (r) is greater than or equal to the minimum spacing (r_m) as specified in Table 1B; (i.e., no overlapping failure cones).
- The closest edge distance (m) is greater than or equal to the minimum edge distance for shear (m_v) as specified in Table 1B. Note: $m_v > r_m/2$; $m_v > m_t$
- The bolt embedment depth is greater than or equal to L_d as specified in Table 1B.

The size of Type A anchor bolts is selected such that the design load (T) does not exceed the basic Nominal Design Resistance ($A_s F_y$) values tabulated in Table 2A.

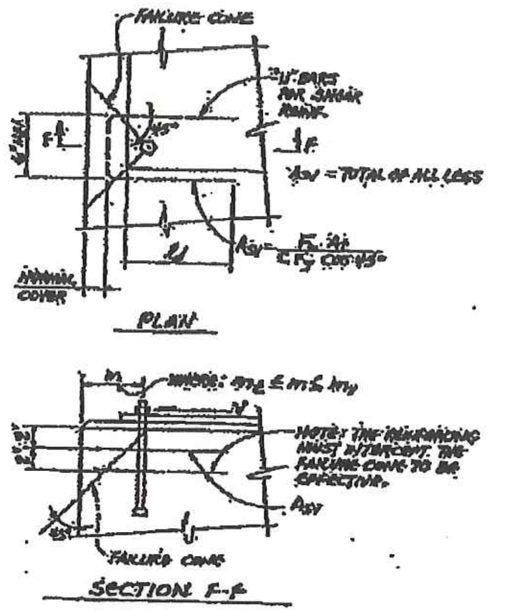


Fig. 3. Shear reinforcement

Type B Anchor Bolts—Anchor bolts are classified as "Type B," or shear reinforcement only, when all of the following apply:

- The closest bolt spacing (r) is greater than or equal to r_m.
- The closest edge distance (m) is greater than or equal to $r_m/2$ but less than m_v. Note: $r_m/2 > m_t$
- The bolt embedment depth is greater than or equal to L_d.

The size of Type B anchor bolts is selected as per Type A anchor bolts. In addition, shear reinforcement (A_{sv}) is provided on both sides of any critical plane of potential failure (see Fig. 3). The total area of horizontal shear reinforcing steel (A_{sv}) is determined as follows:

$$A_{sv} = \frac{F_{ut} A_t}{C F_y \cos 45}$$

where F_y is the specified minimum yield strength of the reinforcing steel.

Type C Anchor Bolts—Anchor bolts are classified as Type C, or shear reinforcement plus overlapping failure cone considerations, when all the following apply:

- The closest bolt spacing (r) is less than r_m.
- The closest edge distance (m) is greater than or equal to m_t and less than m_v. Note: $m_t < r_m/2$

Table 3. Standard Anchor Bolt Tensile Capacities

Bolt Diameter d (in.)	Tensile Stress Area A_t (in.²)	F_uA_t (kips)				
		$F_y = 58$ ksi	$F_y = 90$ ksi	$F_y = 105$ ksi	$F_y = 120$ ksi	$F_y = 150$ ksi
		A36 A307	A449	A325 A449	A325 A449	A687
½	0.142	8.24			17.06	21.3
⅝	0.226	13.11			27.12	33.9
¾	0.334	19.37			40.08	50.1
⅞	0.462	26.80			55.44	69.3
1	0.606	35.15			72.72	90.9
1⅛	0.763	44.25		80.12		114.5
1¼	0.969	56.20		101.7		145.4
1⅜	1.155	66.99		121.3		173.3
1½	1.405	81.49		147.5		210.8
1¾	1.90	110.2	171.0			285.0
2	2.50	145.0	225.0			375.0
2¼	3.25	188.1	292.5			487.5
2½	4.00	232.0	360.0			600.0
2¾	4.93	285.9	443.7			739.5
3	5.97	346.3	537.3			895.5

- The bolt embedment depth must be determined by considering the effect of overlapping concrete tensile stress cones (see Fig. 2). Note: L_d (required) > L_d as tabulated in Table 1B.
- Under no condition will the closest bolt edge distance be less than m_t or 4 in.

The size of Type C anchor bolts is selected as per Type A anchor bolts. Shear reinforcement is provided as per Type B anchor bolts. Also, the bolt embedment depth is calculated as follows:

- First, calculate the effective concrete tensile stress area A_e (see Fig. 2) based on r, m and an assumed embedment depth greater than L_d. The effective concrete tensile stress area (A_e) is the projected area bounded by the intersection between 45 degree lines radiating from the edge of the bolt head and the concrete surface at which the loads are applied, minus the area of the bolt heads (refer to Fig. 2).
- Then, calculate the pullout strength (U_p), where $4\beta\sqrt{f'_c}$ is, the allowable uniform concrete tensile stress applied over the effective stress area A_e:

$$U_p = [4\beta\sqrt{f'_c}]A_e > F_uA_t$$

- Note that U_p must be greater than or equal to the minimum specified tensile strength (F_uA_t) of the standard anchor bolt as tabulated in Table 3. If U_p is less than F_uA_t, continue to increase the bolt embedment depth until a solution is obtained.
- The tensile strength of the concrete failure cone in a slab or wall is limited by the thickness of concrete and the out-to-out dimensions of the anchors. If 45 degree lines extending from the exterior bolt heads toward the compression face do not intersect within the concrete, then the effective stress area is limited as shown in Fig. 2.

Type D Anchor Bolts—Anchor bolts are classified as Type D, or tension lap with reinforcement, when all the following apply:

- The closest bolt spacing (r) is less than r_m.
- The closest edge distance (m) is greater than or equal to m_t and less than $r_m/2$.
- The required bolt embedment depth is greater than or equal to L_d.
- The projected area of the overlapping concrete tensile stress cones (A_e) are extremely limited, such that failure mechanism is controlled by the reinforced section rather than by the yielding of the anchor bolt steel. Such situations commonly arise in concrete piers.

The size of Type D anchor bolts is selected as per Type A anchor bolts. Shear reinforcement is provided as per Type B anchor bolts. Additional tension reinforcement is provided as follows:

- Additional tension reinforcement is provided by concentrically located reinforcing steel (A_{st}), such that the anchor bolts are developed for "full anchorage." Refer to Fig. 4 for the recommended tension reinforcement practice.
- The total area of tension reinforcement (A_{st}) as determined by the following equation is developed on

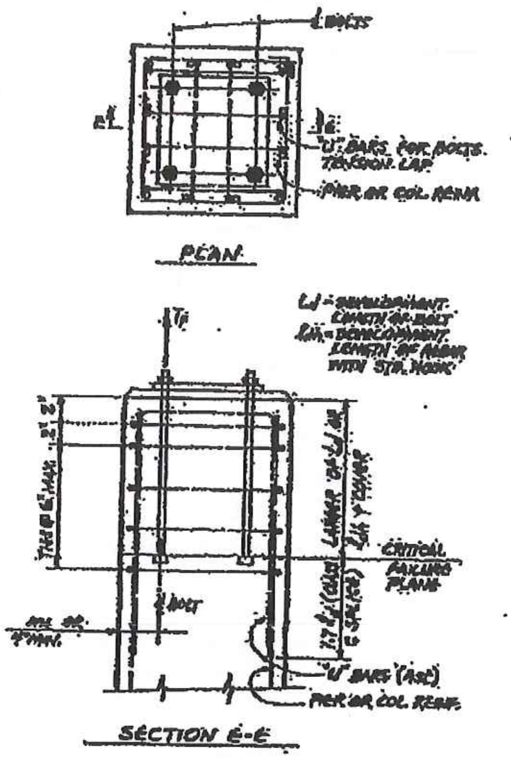


Fig. 4. Tension lap

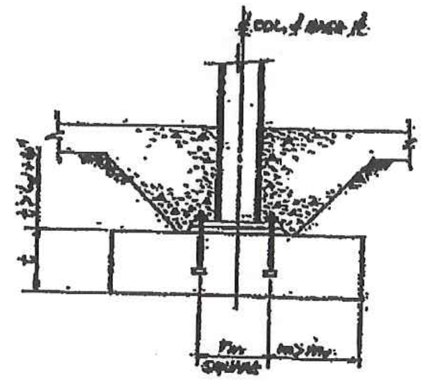


Fig. 5. Example 1: Type A anchor bolt

both sides of the critical plane of potential failure:

$$A_{st} = nF_uA_t/F_y$$

where

n = total number of bolts in the bolt group
F_y = minimum yield strength of reinforcing steel

NUMERICAL EXAMPLES

The application of the criteria presented in this paper is illustrated by the following three example problems. The examples demonstrate Type A and D anchor bolts. An example is also presented for a column base plate for which special attention is given to concrete strength and anchor bolt head placement.

Example 1: Type A (Isolated Bolt), see Fig. 5

Design Data:

$T_F = 35$ kips
$V_i = 15$ kips } DL + LL + WL
$f'_c = 3000$ psi
$SIF = 1.33$; $\alpha = 1/SIF = 0.75$
$\phi = 0.55$ (working stress design)
$C = 1.85$ (grouted base plate)

Design:

$$T = \left[\frac{CV_i + T_F}{\phi}\right]\alpha = \left[\frac{1.85(15) + 35}{0.55}\right]0.75 = 86 \text{ kips}$$

Refer to Table 2A and select $1\frac{3}{8}$-in. dia. A325 bolts:

$A_tF_y = 93.6$ kips > 86 kips.

Use $1\frac{3}{8}$-in. dia. A325 bolts; $r_m = 33$ in. and $L_d = 24$ in.

Example 2: Type D (Bolts in a Confined Pier), see Figs. 6 and 7

Design Data:

Design anchor bolts for cylindrical heater foundation.

For empty + wind load combination:

$T_F = 1$ kips; $V_i = 3$ kips
$F_y = 60$ ksi; $f'_c = 3000$ psi
$SIF = 1.0$; $\alpha = 1.0$
$r = 12$; $m = 4$
$\phi = 0.55$ (working stress design)
$C = 1.85$ (grouted base plate)

Design:

$$T = \left[\frac{CV_i + T_F}{\phi}\right]\alpha\left[\frac{1.85(3) + 1}{0.55}\right] = 11.9 \text{ kips}$$

From Table 2A, for $\frac{3}{4}$-in. dia. A307 anchor bolt:

$A_tF_y = 12.02$ kips ≥ 11.9 kips

$r = 12$ in. $\leq r_m = 12$ in.

$m_t \leq m < m_u$, where $m_t = 4$ in.

$L_d = 9$ in.

$F_uA_t = 19{,}370$ lbs (see Table 3)

$$A_e \text{ (required)} = \frac{f_{ut}A_t}{4\beta\sqrt{f'_c}} = \frac{19{,}370}{4(0.65)\sqrt{3000}} = 136 \text{ sq. in.}$$

$A_e = 10^2 = 100$ sq. in. < 136 sq. in. n.g.

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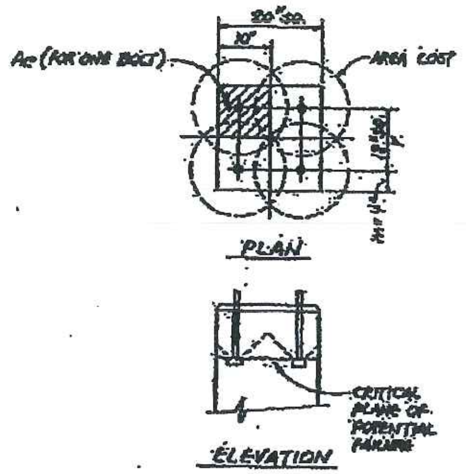

Fig. 6. Example 2: Type D anchor bolt.

Increase pier size to 24 in. square, (to avoid placement of tension reinforcement), such that:

$A_e = 12^2 = 144$ sq. in. > 136 sq. in. o.k.

Next, check the reinforced section and provide tension lap reinforcement.

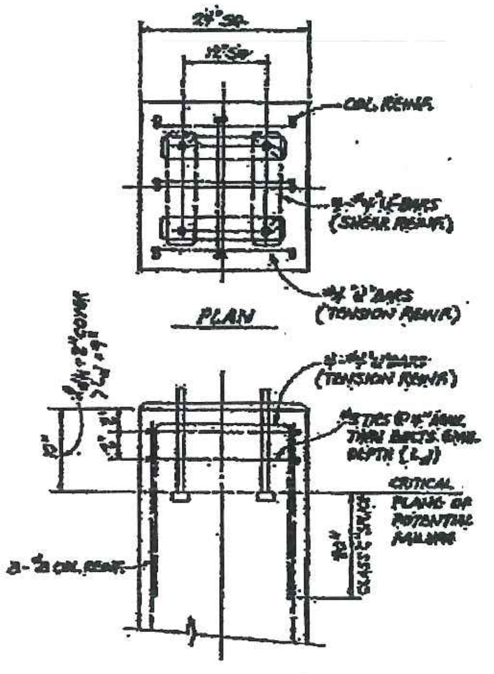

Fig. 7. Example 2: Pier for Type D anchor bolt

Thus, we have a Type D anchor bolt.

$$A_{st} = \frac{nF_u A_t}{F_y} = \frac{4(19.37)}{(60)}$$

$= 1.29$ sq. in. < 1.60 sq. in. (8-#4 bars)

Use 4-#4 U-bars.

Shear reinforcement must also be provided.

$$A_{sv} = \frac{F_u A_t}{GF_y \cos 45°} = \frac{19.37}{(1.85)(60)(.707)}$$

$= 0.25$ sq. in. < 0.40 sq. in. (1-#4 U-bar).

Use: 1-#4 U-bar in each direction.

Example 3: (See Figs. 8 and 9)

Design:

$A_e \cong \pi r^2 = \pi(28)^2 = 2463$ in.^{2}

$U_p = 4\beta\sqrt{f'_c}\, A_e \geq F_u A_t$

$= 4(.85)4000\,(2463) = 529{,}630$ lbs

$F_u A_t = 110{,}200(4) = 440{,}600$ lbs $< 529{,}630$ lbs (see Table 3)

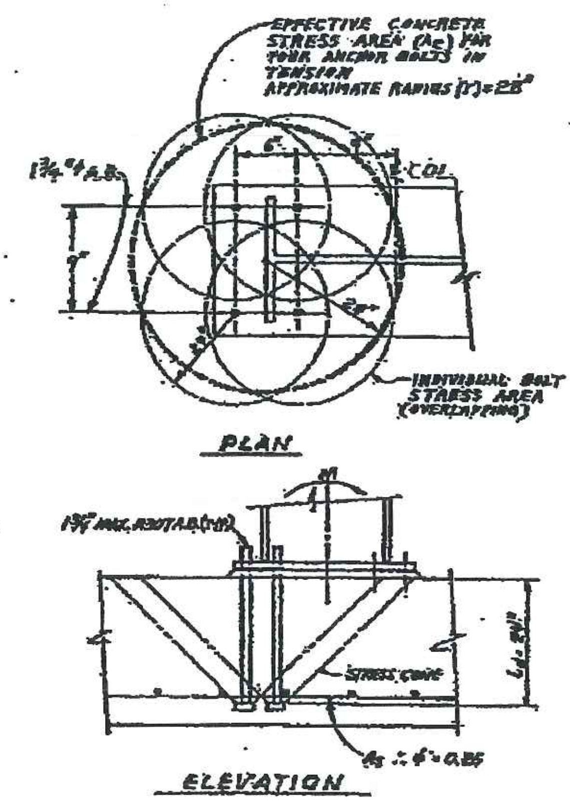

Fig. 8. Example 3: Column base plate.

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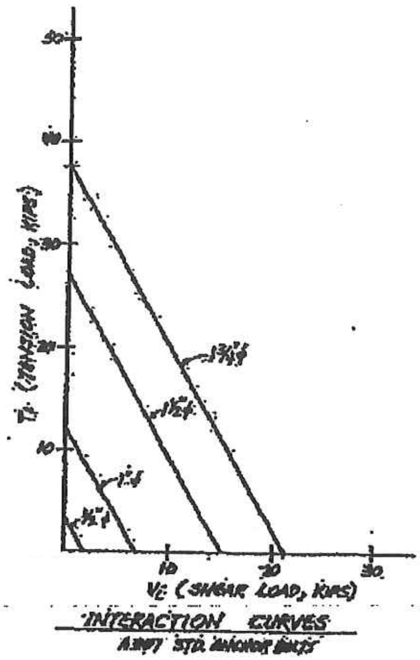

Fig. 9. Example 3: Interaction curves

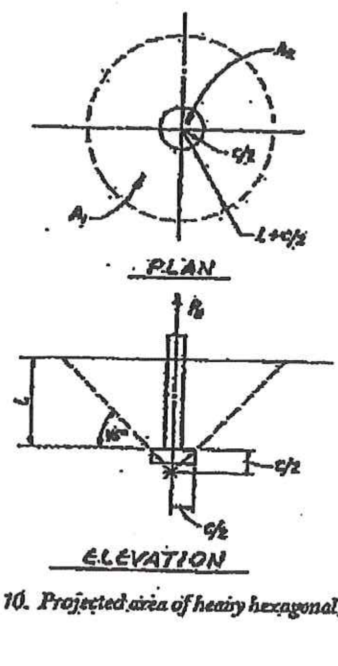

Fig. 10. Projected area of heavy hexagonal head

Therefore, 4-1¾-in. maximum diameter bolts may be used.

Note: $L_d = 24$ in. not adequate if $f'_c = 3000$ psi and $\beta = 0.65$, i.e., anchor bolt head *within* far face reinforcement.

$$A_tF_y \geq T = \left[\frac{CV_t + T_F}{\phi_t}\right]\alpha$$

$$T\phi = \phi A_tF_y = 0.55A_tF_y = CV_t + T_f$$

$$C = 1.85;\ \alpha = 1.0$$

$$\phi = 0.55 \text{(WSD)}$$

$$T = A_tF_y \text{ (Table 2A)}$$

Anchor Bolt Working Stress Loads: See Fig. 9 for plot.

A307 Bolt Dia. (in.)	$0.55A_tF_y$	V_t	T_F
½	2.82	0 1.52	2.82 0
1	12.00	0 6.49	12.00 0
1½	27.82	0 15.04	27.82 0
1¾	37.62	0 20.34	37.62 0

NOMENCLATURE

A_e = Effective projected stress area to which the allowable uniform concrete tensile stress is applied to determine the pullout strength of concrete

A_{st} = Total area of reinforcing steel across a potential tension failure plane(s)

A_{sv} = Total area of reinforcing steel across a potential shear failure plane(s)

A_t = Tensile stress area of anchorage per AISC[4]

C = Shear coefficient applied to standard anchors which accounts for effects of various shear failure surfaces

= 1.10 when steel plates are embedded with exposed surface flush with concrete surface

= 1.25 when steel plates are recessed in grout with bottom of plate in concrete surface

= 1.85 when steel plates are supported on grout mortar with exposed surface exterior to concrete surface

c = Equivalent circle for hex head

d = Nominal diameter of a bolt or plain bar

f'_c = Specified compressive strength of concrete

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F_y = Minimum specified yield strength of steel or rebar as tabulated below:

F_y (ksi)	ASTM	Bolt Diameter (in.)
36	A307	All
92	A325	½ to 1, incl.
81	A325	Over 1 to 1½, incl.
92	A449	½ to 1, incl.
81	A449	Over 1 to 1½, incl.
58	A449	Over 1½ to 3, incl.
105	A687	⅝ to 3, incl.
60	A615	Type S, Grade 60 Rebar
40	A615	Grade 40 Rebar

F_u = Minimum specified tensile strength of steel as tabulated below:

F_u (ksi)	ASTM	Bolt Diameter (in.)
58	A307	All
120	A325	½ to 1, incl.
105	A325	Over 1 to 1½, incl.
120	A449	1/2 to 1, incl.
105	A449	Over to 1 to 1½, incl.
90	A449	Over 1½ to 3, incl.
150	A687	⅝ to 3, incl.

h = Thickness of a concrete slab or wall

L_d = Minimum embedded length required to fully develop the tensile strength of an anchor bolt

l_d = Basic development length for reinforcement

l_{dh} = Development length of reinforcement with a standard hook

m = Edge distance from the center of an anchor to the edge of concrete

m_t = Minimum edge distance to prevent failure due to lateral bursting forces at a standard anchor bolt head

m_v = Minimum edge distance to develop the full tensile capacity of an anchor bolt in shear within additional reinforcement when the shear load acts toward the free edge

n = Number of bolts in a bolt group

PF = Probability Factor

r = Spacing of multiple anchors

r_m = Minimum spacing of multiple anchor bolts

SIF = Stress Increase Factor

T = Total effective anchor bolt design tension load due to bending and direct load

T_P = Tension load acting on an individual anchor bolt or wedge anchor

U_p = Pullout strength of concrete equal to the tensile capacity of the concrete failure cone

V = Total shear in an anchorage

V_i = Shear load acting on an individual anchor

ϕ = Capacity reduction factor
= 0.90 for factored design loads under Ultimate Strength Design (USD) for steel tensile stress
= 0.55 for service design loads under Working Stress Design (WSD); complies with AISC allowable F_t values

μ = Coefficient of friction

α = Probability Factor (PF) or reciprocal of the stress increase factor ($1/SIF$)

β = Concrete tensile stress reduction factor
= 0.65 for concrete tensile stress when embedded anchor head is *within* far face reinforcement
= 0.85 for concrete tensile stress when embedded anchor head is *beyond* the far face reinforcement

ACKNOWLEDGMENTS

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13. *Lee, D. W. and J. E. Breen* "Factors Affecting Anchor Bolt Development" *Research Report 88-1F, Project 3-5-65-88; Cooperative Highway Research Program with Texas Highway Department and U.S. Bureau of Public Roads, Center for Highway Research, University of Texas, Austin, Aug. 1966.*
14. *McMackin, P.; R. Slutter and J. Fisher* "Headed Steel Anchors Under Combined Loading" *Engineering Journal, American Institute of Steel Construction, Vol. 10, No. 2, 1973.*
15. *PCI* "Design Handbook—Precast Prestressed Concrete" *Second Edition, 1974.*
16. *Swirsky, R. A. et al* "Lateral Resistance of Anchor Bolts Installed in Concrete" *California Department of Transportation, Sacramento, U.S. Department of Commerce National Technical Information Service PB80-116189, May 1979.*
17. *TVA.* "Anchorage to Concrete" *Tennessee Valley Authority Division of Engineering Design, Thermal Power Engineering—Report No. CEB 75-32, Dec. 1, 1975.*

APPENDIX A.
MINIMUM SPACING AND EMBEDMENT

An equivalent circle is assumed equal to the projected area of a heavy hexagonal head (see Fig. 10).

$$A_{hex} = \left(\frac{\sqrt{3}}{2}\right) F^2 = 0.866F^2$$

$$A_{circle} = \pi C^2/4$$

$$0.866F^2 = \pi C^2/4$$

$$C = \sqrt{\frac{0.866F^2(4)}{\pi}} = 1.05F$$

Tensile stress area $A_t = A_1 - A_2$

$$= \pi (L + C/2)^2 - \pi (C/2)^2$$
$$= \pi [L^2 + CL + C^2/4 - C^2/4]$$
$$= \pi [L^2 + CL]$$

$$U_p = A_t [4\beta \sqrt{f'_c} \quad (\text{assume } \beta = 0.65)$$
$$= \pi [L^2 + CL][4(0.65)\sqrt{3000}]$$
$$= \pi [L^2 + CL]142$$
$$= 447(L^2 + CL)$$

Also, $U_p = F_u A_t$, in pounds (see Table 3).

Therefore,

$$0 = 447AL^2 + 447ACL - F_u A_t$$

$$0 = L^2 + CL - (F_u A_t / 447A)$$

$$L = \frac{-C \pm \sqrt{C^2 + 4\left[\frac{F_u A_t}{447}\right]}}{2}$$

$$= \frac{\sqrt{C^2 + \left[\frac{F_u A_t}{112}\right]} - C}{2}$$

See Table 4 for tabulated values. The design criteria are as follows:

1. Minimum spacing of bolts (τ_m):
   For A307: $2 \times 8.0d = 16d$
   For A325/A449: $2 \times 12.0d = 24d$
   For A687: $2 \times 14.0d = 28d$

Table 4. Tabulated Values of L

Bolt Diameter d (in.)	Tensile Stress Area A_t (in.²)	Heavy Hex Width Across Flats F (in.)	Eff. Dia. C (in.)	A36, A307		A325, A449		A687	
				L (in.)	*L/d	L (in.)	*L/d	L (in.)	*L/d
½	0.142	0.875	0.92	3.9	7.8	5.8	11.6	6.5	12.9
⅝	0.226								
¾	0.334	1.25	1.32	6.0	8.0	8.9	11.9	10.0	13.3
⅞	0.462								
1	0.606	1.625	1.71	8.1	8.1	12.0	12.0	13.4	13.4
1¼	0.969								
1½	1.41	2.375	2.50	12.4	8.3	17.0	11.4	20.5	13.6
1¾	1.90								
2	2.50	3.125	3.28	16.5	8.3	22.7	11.4	27.3	13.7
2¼	3.25								
2½	4.00	3.875	4.07	20.9	8.4	28.7	11.5	34.6	13.8
2 ¾	4.93								
3	5.97	4.625	4.86	25.5	8.5	35.1	11.7	42.3	14.1

* To ensure ductile failure, use the value of L/d obtained by multiplying the largest L/d value in each column by an arbitrary factor of safety of 1.33:
For A36, A307: $L/d = 1.33\ (8.5) = 12$
For A325, A449: $L/d = 1.33\ (12.0) = 16$
For A687: $L/d = 1.33\ (14.1) = 19$

2. Formula for embedment length (L_d):

$$L_d = 12d\sqrt{\frac{f_u}{58000}}, \quad \text{where } F_u \text{ is in ksi.}$$

3. Embedment length (L_d):

   For A307: $L_d = 12d$

   For A325/A449: $L_d = 17d$

   For A687: $L_d = 19d$.

4. Values are tabulated in Table 2.

APPENDIX B: BOLT TENSION/ SHEAR INTERACTION EQUATIONS

The area of steel required for tension and shear is considered additive.

$$A_v = \frac{\alpha CV}{F_v} = \text{area of steel required for shear}$$

$$A_T = \frac{\alpha T_F}{F_A} = \text{area of steel required for tension}$$

where

F_v = allowable shear stress
F_A = allowable tension stress
α = Probability factor (PF) or reciprocal of the stress increase factor ($1/SIF$).
Note: $\alpha \leq 1.0$.

$$A_v + A_T = A_t$$

where A_t = tensile stress area of anchorage

$$\frac{\alpha CV}{F_v} + \frac{\alpha TF}{F_A} = A_t$$

$$\frac{CV}{F_v A_t} + \frac{T_F}{F_A A_t} = \frac{1}{\alpha}$$

The shear force (V) causes a crushing/bearing failure near the surface and translates the shear load into an effective tension load in the anchorage.

$$F_v = F_A$$

$$F_v A_t = F_A A_t = \phi T$$

$$\frac{CV}{\phi T} + \frac{T_F}{\phi T} = \frac{1}{\alpha}$$

$$T = \left[\frac{CV + T_F}{\phi}\right]\alpha$$

Note that A_T may be solved for as follows:

$$\frac{\alpha CV}{F_v} + \frac{\alpha T_F}{F_A} = A_t$$

$$F_v = F_A = \phi F_y$$

$$A_t = \left[\frac{CV + T_F}{\phi F_y}\right]\alpha$$

Expressed as an interaction equation:

$$\frac{CV}{\phi F_y A_t} + \frac{F}{\phi F_y A_t} \leq \frac{1}{\alpha}$$

APPENDIX C. PROBABILITY-BASED LIMIT STATES DESIGN (PBLSD)

1. The PBLSD design criterion is expressed in general form as follows:

   Design Resistance ≥ Effect of Design Loads

   In equation form: $\phi R \geq \gamma_c \sum_{k=1}^{j} Q_k \gamma_k$

   where

   ϕ = resistance factor, less than 1.0, accounts for uncertainties in material strength
   R = nominal design resistance (capacity), equal to the plastic strength of a structural member
   γ_c = analysis factor
   γ_k = load factor, normally greater than 1.0, and provides for load variations
   Q_k = nominal design load effect
   $\sum_{k=1}^{j}$ = denotes the combined load effects from various causes

2. The PBLSD uses the concept of "limit state" design. The nominal resistance (R) is always related to a specific "limit state." Two classes of limit states are pertinent to structural design: the "ultimate limit state" and the "serviceability or working limit state." Violation of the "ultimate limit state" involves loss of all or parts of the structure mechanism. "Serviceability limit state" involves excessive deflection, excessive vibration and gross yielding.

3. The anchor bolt design equation expressed in PBLSD form may be derived as follows:

   $$\phi R \geq \gamma_c \sum_{k=1}^{j} Q_k \gamma_k$$

   Let $R = F_y A_t$

   where

   F_y = minimum yield strength of steel
   A_t = bolt tensile area

   Let $\gamma_c = \alpha$

   Let $\sum_{k=1}^{j} \gamma_k Q_k = CV_i + T_F$

   (the combined effect of tension and shear loads as derived in Appendix B.)

   where

   C = Shear coefficient

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$V_i = \gamma_1 V_1 + \gamma_2 V_2 + \ldots \gamma_k V_k$
$T_F = \gamma_1 T_1 + \gamma_2 T_2 + \ldots \gamma_k T_k$
γ_1 = Load factor for load case number 1
γ_2 = Load factor for load case number 2

By substitution: $\phi F_y A_t \geq [CV_i + T_F]\alpha$

$$F_y A_t \geq \left[\frac{CV_i + T_F}{\phi}\right]\alpha = T$$

where $F_y A_t$ values are tabulated in Table 2A.

Note: $\phi = 0.90$ is a resistance factor which accounts for uncertainties in material strength (USD).
$\phi = 0.55$ is a resistance factor which converts the yield capacity to working loads (WSD)

SR520 BRIDGE TREATMENT TANK
PILE FOUNDATION DESIGN
January, 2011
By: AM

Required: Design slab and piles for treatment tank.

Location: Aberdeen, WA

References:

1. GeoTechnical Report by Shannon & Wilson, attached
2. 2009 IBC
3. ASCE7 - 05, "Minimum Design Loads for Buildings..."
4. ACI 318-05, "Building Code Requirements for Structural Concrete"
5. AISC "Manual of Steel Construction", Ed. 13
6. Sketch of existing tank (attached)

Given:

$f'_c := 4000 \cdot psi$ — concrete compressive strength

$F_y := 60000 \cdot psi$ — ASTM A615, Grade 60 reinforcing bars

$$W_{fluid} := (24000 \cdot gal) \cdot \left(8.34 \cdot \frac{lb}{gal}\right) \qquad W_{fluid} = 200 \cdot k$$

$$W_{tank} := 10.0 \cdot k$$

$$W_{slab} := 150 \cdot \frac{lb}{ft^3} \cdot \left[18 \cdot in\,(19.0 \cdot ft)^2 + (1.5 \cdot ft)(4) \cdot (19.0 \cdot ft) \cdot (12 \cdot in)\right] - 100 \cdot \frac{lb}{ft^3} \cdot \left(114 \cdot ft^3 + 180.5 \cdot ft^3\right)$$

$$W_{slab} = 69 \cdot k$$

Wind Loading

$h := 31 \cdot ft$ $\quad D := 12.5 \cdot ft$

$K_z := 1.17$ — Ref. 3, Tbl. 6-3, Exposure D

$K_d := 0.95$ — Tbl. 6-4

$K_{zt} := 1.0$

$V := 90$ — for temporary structure at water's edge

$I := 1.0$

$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I \cdot psf$ $\quad q_z = 23 \frac{lb}{ft^2}$ — Eqn. 6-15

$G := 0.85$

$\frac{h}{D} = 2.48$ $\quad C_f := 0.6$ — Fig. 6-21

$A_f := h \cdot D$ $\quad A_f = 387.5 ft^2$

$F_w := q_z \cdot G \cdot C_f \cdot A_f$ — Eqn. 6-28

$F_w = 4555 lb$

Overturning - empty tank

$M_r := 0.9(W_{tank}) \cdot \frac{D}{2}$ $\quad M_r = 56.25 \cdot ft \cdot k$ — Ref. 3, 2.3.2, Eqn. 6

$M_{OT} := 1.6 F_w \cdot \frac{h}{2}$ $\quad M_{OT} = 113 \cdot ft \cdot k$

$T := \frac{M_{OT} - M_r}{2D}$ $\quad T = 2268 lb$ — 50% factor to account for ring of holddowns.

$\boxed{\text{Need anchor bolt design}}$

Anchor Bolts

See attached Hilti software analysis.

Soil Pressure - full tank

$$p := \frac{1.2\left(W_{slab} + W_{tank} + W_{fluid}\right)}{(19.0 \cdot ft)^2} + \frac{0.8T}{\frac{(19.0 \cdot ft)^2}{2}}$$

$p = 938 \cdot psf$

Ref. 3, 2.3.2, Eqn. 3

$$F_{pile} := \frac{p \cdot (19.0 \cdot ft)^2}{2(0.65)}$$

$F_{pile} = 260 \cdot k$

Ref. 1, Fig. 2

Use 2 steel piles, 18" diam, 3/8" thick, 122' below ground surface.

Slab Reinforcement

Try #5 @ 12" o.c.

$d := 18 \cdot in$ $\quad b := 12 \cdot in$ $\quad A_s := 2 \cdot 0.307 \cdot in^2$

$$\rho := \frac{A_s}{b \cdot d}$$

$\rho = 0.00284 \quad > \quad 0.0018$

Ref. 4, 7.12.2.1

OK for shrinkage and temperature

Holddowns

Bending

$$S := \frac{6.0\cdot in\cdot(0.375\cdot in)^2}{6}$$ $S = 0.14\cdot in^3$

$$M_{hd} := 1.6T\cdot\frac{2.5\cdot in}{2}$$ $M_{hd} = 4.54\cdot in\cdot k$

$Z := S\cdot\frac{6}{4}$ $F_y\cdot Z = 12.7\cdot in\cdot k$ < $1.6\cdot F_y\cdot S = 13.5\cdot in\cdot k$ > M_{hd} OK

Ref. 5, F11-1

Shear

$\frac{1.6\cdot F_w}{(.875\cdot in\cdot 2)\cdot(0.375\cdot in)} = 11.11\cdot ksi$ < $(0.90)\cdot(0.60)\cdot F_y = 32.4\cdot ksi$ OK

Ref. 5, G2-1

Head Plate Punching Shear

$d := 11.0\cdot in$ depth of head plate

$b_o := 4\cdot(d + 24\cdot in)$ $b_o = 11.67 ft$

$V_c := 4\cdot\sqrt{f'_c\cdot psi}\cdot b_o\cdot d$ $V_c = 390\cdot k$

$\phi := 0.75$

$\phi\cdot V_c = 292\cdot k$ > pile capacity

OK

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PROFIS Anchor 2.1.1

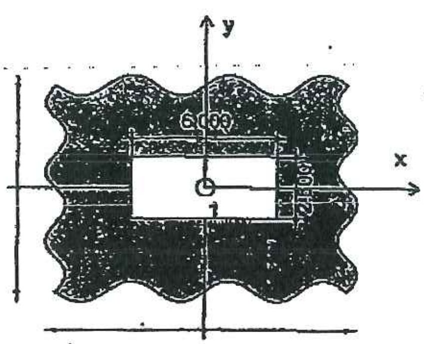
Company:		Page:	1
Specifier:		Project:	520 Treatment Tank
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	-|-	Date:	1/11/2011
E-Mail:			

Specifier's comments:

Input data

Anchor type and diameter:	HIT-RE 500-SD + HAS B7, 5/8
Effective embedment depth:	$h_{ef,act}$ = 3.116 in. ($h_{ef,limit}$ = 12.500 in.)
Material:	ASTM A 193 Grade B7
Evaluation Service Report:	ESR 2322
Issued I Valid:	4/1/2010 | -
Proof:	design method ACI 318 / AC308
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 6.000 x 2.500 x 0.500 in. (Recommended plate thickness: not calculated)
Profile:	no profile
Base material:	cracked concrete , 3000, f_c' = 3000 psi; h = 18.000 in., Temp. short/long: 32/32°F
Installation:	hammer drilled hole, installation condition: dry
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

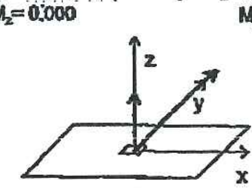
Geometry [in.]



Loading [lb, in.-lb]

Governing loads

N = 2268 V_y = 4555
M_z = 0.000 M_y = 0.000



Eccentricity (structural section) [in.]

e_x = 0.000 V_x = 0
e_y = 0.000 M_x = 0.000

Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb] Load	Capacity	Utilization [%] β_N/β_V	Status
Tension	Concrete Breakout Strength	2268	3328	68 / -	OK
Shear	Pryout Strength	4555	7168	- / 64	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.682	0.636	5/3	100	OK

Warnings

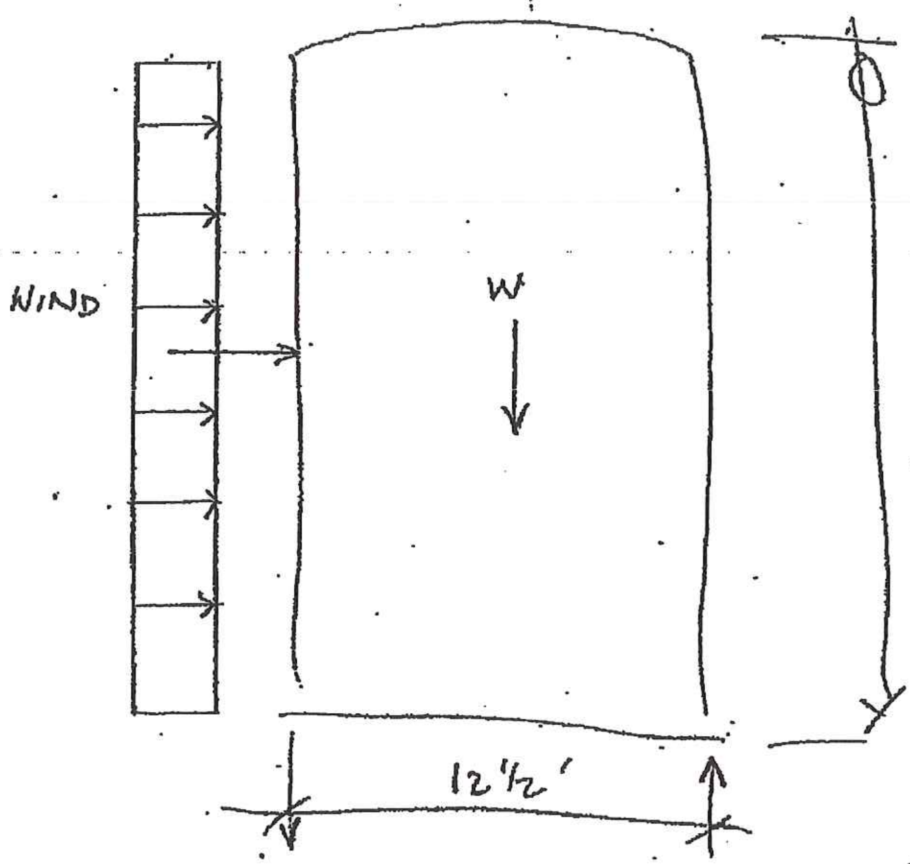
- Please consider all details and hints/warnings given in the detailed report!

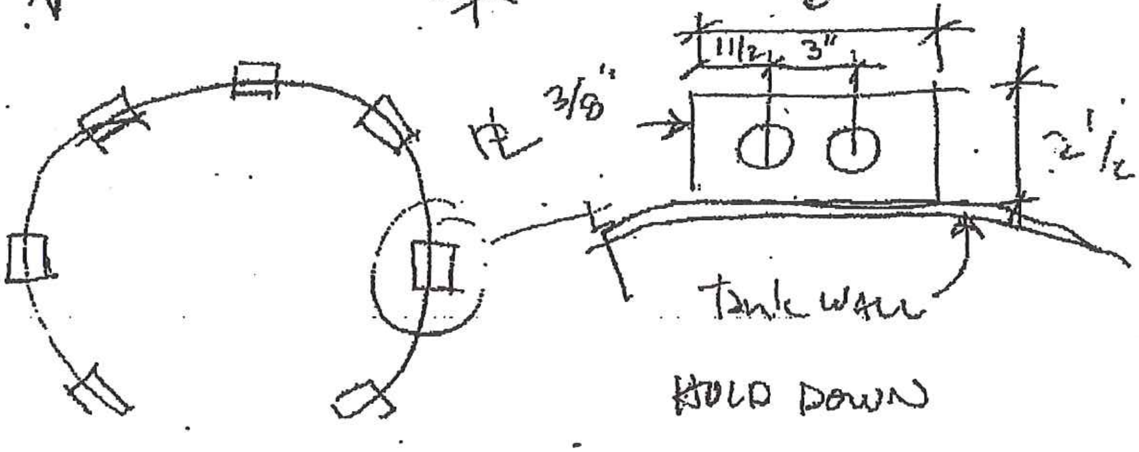
Fastening meets the design criteria!

Input data and results must be checked for agreement with the existing conditions and for plausibility!

520 Fiberglass Treatment Tank

- OWNER SAID TANK EMPTY WEIGHED ~ 10,000 lbs
- 24,000 gal capacity
- Fiberglass w/ Insulation





2-6

SR520 BRIDGE PLANT SILO
PILE FOUNDATION DESIGN
January, 2011
By: AM

Required: Design slab and piles for cement silo.

Location: Aberdeen, WA

References:

1. GeoTechnical Report by Shannon & Wilson, attached
2. 2009 IBC
3. ASCE7 - 05, "Minimum Design Loads for Buildings..."
4. ACI 318-05, "Building Code Requirements for Structural Concrete"
5. AISC "Manual of Steel Construction", Ed. 13
6. Sketch of existing silo (attached)

Given:

$f'_c := 4000 \cdot psi$ — concrete compressive strength

$F_y := 60000 \cdot psi$ — ASTM A615, Grade 60 reinforcing bars

$W_{cement} := 110 \cdot k$

$W_{tank} := 18.0 \cdot k$

$W_{slab} := 150 \cdot \frac{lb}{ft^2} \cdot \left[(19.0 \cdot ft)^2 + (1.5 \cdot ft)(4) \cdot (19.0 \cdot ft)\right]$ $\quad W_{slab} = 71.25 \cdot k$

Wind Loading

$h := 31.9 \cdot ft$ $D := 8.5 \cdot ft$

$K_z := 1.17$ Ref. 3, Tbl. 6-3, Exposure D

$K_d := 0.95$ Tbl. 6-4

$K_{zt} := 1.0$

$V := 90$ for temporary structure at water's edge

$I := 1.0$

$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I \cdot psf$ $q_z = 23 \frac{lb}{ft^2}$ Eqn. 6-15

$G := 0.85$

$\frac{h}{D} = 3.75$ $C_f := 0.6$ Fig. 6-21

$A_f := h \cdot D$ $A_f = 271.15 ft^2$

$F_w := q_z \cdot G \cdot C_f \cdot A_f$ Eqn. 6-28

$F_w = 3187 lb$

Overturning - empty tank

$M_r := 0.9(W_{tank}) \cdot \left(\frac{94 \cdot in}{2}\right)$ $M_r = 63.5 \cdot ft \cdot k$ Ref. 3, 2.3.2, Eqn. 6

$M_{OT} := 1.6 F_w \cdot \left[7.8 \cdot ft + \left(\frac{24.08 \cdot ft}{2}\right)\right]$ $M_{OT} = 101 \cdot ft \cdot k$

$T := \frac{M_{OT} - M_r}{94 \cdot in}$ $T = 4816 lb$

Need anchor bolt design

Anchor Bolts

See attached Hilti software analysis.

Soil Pressure - full tank

$$p := \frac{1.2(W_{slab} + W_{tank} + W_{cement})}{(19.0 \cdot ft)^2} + \frac{0.8T}{\frac{(19.0 \cdot ft)^2}{2}}$$

$p = 684 \cdot psf$

Ref. 3, 2.3.2, Eqn. 3

$$F_{pile} := \frac{p \cdot (19.0 \cdot ft)^2}{2(0.65)}$$

$F_{pile} = 190 \cdot k$

Ref. 1, Fig. 2

Use 2 steel piles, 18" diam, 3/8" thick, 122' below ground surface.

Slab Reinforcement

Try #5 @ 12" o.c. each face.

$d := 12 \cdot in$ $\quad b := 12 \cdot in$ $\quad A_s := 2 \cdot 0.307 \cdot in^2$

$$\rho := \frac{A_s}{b \cdot d}$$

$\rho = 0.00426 \quad > \quad 0.0018$

Ref. 4, 7.12.2.1

OK for shrinkage and temperature

Head Plate Punching Shear

$d := 8.0 \cdot in$ depth of head plate

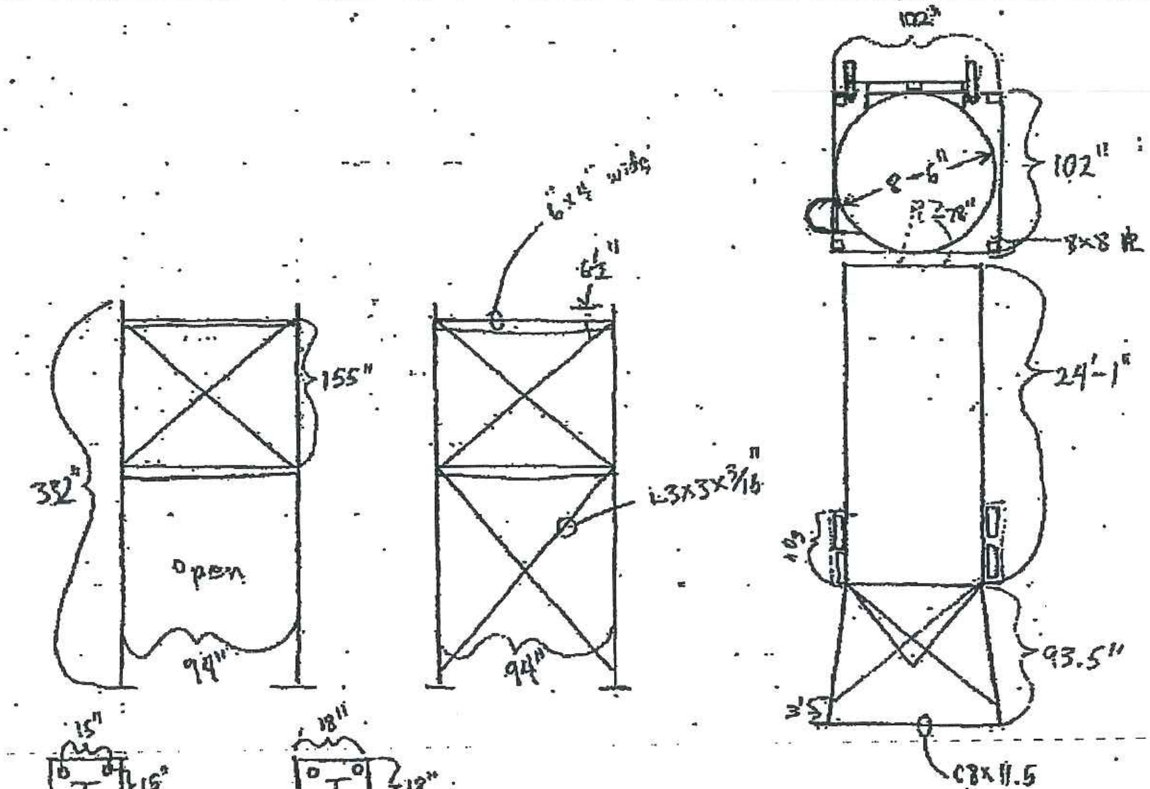
$b_o := 4 \cdot (d + 24 \cdot in)$ $b_o = 10.67 ft$

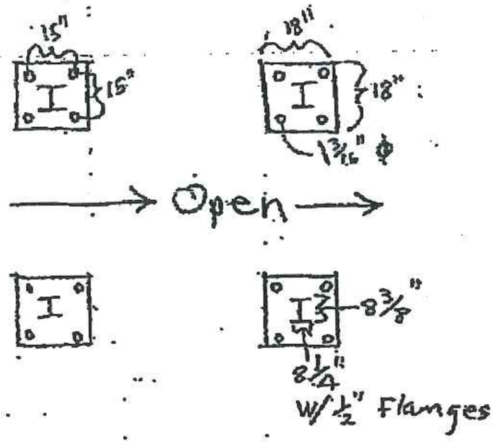
$V_c := 4 \cdot \sqrt{f_c \cdot psi} \cdot b_o \cdot d$ $V_c = 259 \cdot k$

$\phi := 0.75$

$\phi \cdot V_c = 194 \cdot k$ > pile capacity

OK





Empty — 18,000 lbs.

Cement - 110,000 lbs.

HILTI

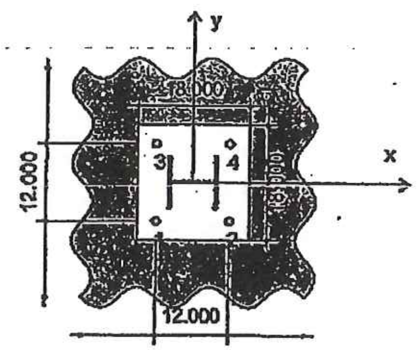
Company:		Page:	1
Specifier:		Project:	520 Bridge
Address:		Sub-Project I Pos. No.:	
Phone I Fax:	- | -	Date:	1/11/2011
E-Mail:			

Specifier's comments:

Input data

Anchor type and diameter:	HIT-RE 500-SD + HAS B7, 1
Effective embedment depth:	$h_{ef,act}$ = 4.000 in. ($h_{ef,limit}$ = 9.750 in.)
Material:	ASTM A 193 Grade B7
Evaluation Service Report::	ESR 2322
Issued I Valid:	4/1/2010 | -
Proof:	design method ACI 318 / AC308
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 18.000 x 18.000 x 0.500 in. (Recommended plate thickness: not calculated)
Profile	W shape (AISC); (L x W x T x FT) = 8.000 in. x 8.000 in. x 0.285 in. x 0.435 in.
Base material:	cracked concrete , 4000, f_c' = 4000 psi; h = 12.000 in., Temp. short/long: 32/32°F
Installation:	hammer drilled hole, Installation condition: dry
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

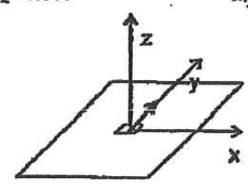
Geometry [in.]



Loading [lb, in.-lb]

Governing loads

N = 2408 V_y = 797
M_z = 0.000 M_y = 0.000



Eccentricity (structural section) [in.]

e_x = 0.000 V_x = 0
e_y = 0.000 M_x = 0.000

Proof I Utilization (Governing Cases)

Loading	Proof	Design values [lb] Load	Capacity	Utilization [%] β_N/β_V	Status
Tension	Concrete Breakout Strength	2408	22364	11 / -	OK
Shear	Pryout Strength	797	48168	- / 2	OK

Loading	β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
Combined tension and shear loads	0.108	0.017	5/3	3	OK

Warnings

- Please consider all details and hints/warnings given in the detailed report!

Fastening meets the design criteria!

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

ALASKA
CALIFORNIA
COLORADO
FLORIDA
MISSOURI
OREGON
WASHINGTON

December 21, 2010

Craig Overly
CalPortland Company
P.O. Box 1730
5975 E Marginal Way S
Seattle, WA 98111

RE: PILE FOUNDATION RECOMMENDATIONS, TEMPORARY CONCRETE BATCH PLANT, SR520 PONTOON CASTING FACILITY, ABERDEEN, WASHINGTON

Dear Mr. Overly,

The purpose of this letter is to provide deep foundation recommendations for the temporary Concrete Batch Plant (CBP) that the CalPortland Company will be constructing for the proposed State Route 520 Pontoon Casting Facility (PCF).

The Washington State Department of Transportation (WSDOT) has contracted Kiewit-General (KG) to construct a casting basin facility to fabricate 33 concrete pontoons within the 55-acre Aberdeen Log Yard property at 400 East Terminal Way, Aberdeen Washington. The property is located within Aberdeen tidelands on the north shore of Grays Harbor near the lower reach of the Chehalis River. The CBP will be located in the northwestern portion of the PCF and the approximate location of the temporary CBP is shown on Figure 1.

Historically, two sawmills operated on the site in the last century, but since 1971 the site has been primarily used for log storage. All former sawmill-related structures have been demolished. Between 1971 and 1981, the shoreline was extended to the south through backfill placement with sediments dredged from the Chehalis River, accumulated wood waste, and other fill material.

The temporary CBP structure will be supported on a 6-inch thick by 20 feet wide by 80 feet long reinforce concrete slab. Based on discussions with the structural engineer the slab will have a uniform pressure of approximately 1,500 pound per square foot (psf).

The following sections describe the analyses, geotechnical recommendations, and construction considerations for foundation support of the temporary CBP.

400 NORTH 34th STREET – SUITE 100
PO BOX 300303
SEATTLE, WA 98103
206-632-8020 FAX 206-695-6777
TDD: 1-800-833-6388
www.shannonwilson.com

SUBSURFACE CONDITIONS

We reviewed the results of the explorations located within the general limits of the proposed temporary CBP. In general, the subsurface conditions consist of fill with wood and occasional concrete debris to a depth of about 10 to 15 feet bgs. In some explorations across the site, the thickness of the wood debris appeared to be extensive, while in others it may be limited to less than a foot. The depth of wood debris noted in the logs near the proposed temporary CBP varied between 1 and 14 feet below ground surface (bgs), with an average depth of about 9 feet bgs.

Very soft to medium stiff silt of medium to high plasticity underlies the fill to an elevation of about -60 to -70 feet MLLW.

Very loose to medium dense, silty sand and medium stiff to stiff silt underlie the surficial silt encountered at the site to about elevation -90 to -110 feet MLLW. Based on a review of the subsurface profiles PCF Geotechnical Baseline Report (GBR) and Shannon & Wilson (2010) the silty sand does not appear to be continuous across the site.

Dense to very dense sand and gravel was encountered below an elevation of about -90 to -110 feet MLLW. The dense to very dense sand and gravel was encountered at elevation -90 and -107 feet MLLW in nearby borings H-1-08 and H-07-09, respectively.

Siltstone was encountered at an elevation of about -185 feet MLLW in boring H-08-09.

AXIAL RESISTANCE

Based on the uniform slab pressure of 1,500 psf and the potential for unsuitable settlement of the soft compressible site soils, we recommend the temporary CBP be supported on deep foundations. Typical pile foundations for the temporary CBP would be either timber piles or steel pipe piles. The basin slab for the proposed PCF will be supported with 18-inch diameter by 3/8-inch thick wall steel pipe piles. Therefore we considered 18-inch diameter by 3/8-inch thick wall steel closed-end pipe piles for support of the temporary CBP.

The recommendations for pile foundation penetrations and capacities are based on theoretical and empirical data, subsurface conditions encountered at the site, engineering judgment, and experience.

Driven pile axial capacities are developed through a combination of side and base resistance. Static axial resistances for the temporary CBP steel pipe piles and timber piles were estimated based on soil types encountered in the borings, relative densities of the soil as determined by SPT blow count, and our experience in similar soil and project conditions. We also considered the results of the March 2010 PCF test pile program and PDA/CAPWAP analyses for estimating

LATERAL RESISTANCE

Lateral loads acting on the structure from wind may be resisted by the lateral resistance provided by the timber or steel pipe piles. The computer programs LPILE^{PLUS} (Reese and Wang, 1997) and Deep Foundation System Analysis Program (DFSAP) (WSDOT, 2006) may be used to evaluate lateral resistance of driven piles and to calculate the magnitude of deflection, shear, and moment along the pile.

Based on subsurface conditions as interpreted from the subsurface explorations the recommended parameters for input into the LPILE^{PLUS} and DFSAP programs under static wind loading conditions are presented in Table 2.

As shown in Table 2, we recommend that the sand and gravel deposits be modeled with the "Reese Sand" constitutive model, which requires a friction angle and modulus of subgrade reaction. The static soil parameters for other layers were estimated based on our review of the consolidated undrained and unconsolidated undrained triaxial tests results, static direct simple shear (DSS) test results, pressuremeter test results, and field vane shear test results, conducted for the PCF and our experience with similar soils.

Group interaction should be considered when evaluating horizontal pile movement for piles with center-to-center spacing less than five times the diameter of the pile. Based on discussions with the structural engineer we assume that the piles for the CBP will have center-to-center spacings greater than five diameters.

CONSTRUCTION CONSIDERATIONS

The pile type and size, estimated pile length, nominal compression resistance, minimum pile driving blow count, minimum ram stroke, and maximum compression stress, as required, are summarized in the table below. We understand that the steel pipe piles and timber piles will be driven using a Delmag D-46 diesel pile driving hammer and the Vulcan No. 1 pile driving hammer, respectively.

The pile driving criteria for the steel pipe pile are based the WEAP analysis using a Delmag D-46 diesel pile driving hammer and our experience with similar projects. The WEAP analysis results are shown graphically in Figure 4. The pile driving criteria for the timber piles is estimated by dividing the nominal resistance (kips) by 4 to achieve a continuous pile driving blow count (blows/foot).

the axial resistance for the steel pipe piles. The PDA/CAPWAP results are presented and summarized in Shannon & Wilson (2010).

These analyses are applicable to a single pile, or pile groups with a center-to-center pile spacing greater than 2.5 diameters. If a pile spacing that is less than 2.5 diameters is selected for design, pile group reduction will be required.

The temporary CBP piles will not be designed for the seismic loading conditions. Therefore, we did not estimate the seismic induced downdrag loads for the temporary CBP piles.

Results of our axial resistance analyses are presented graphically in Figures 2 and 3 in terms of plots of pile penetration versus nominal (unfactored) resistance. Resistances for the 18-inch diameter, closed-end, steel pipe pile would be driving the piles at least 2 feet into the medium dense sand and gravel. A nominal resistance of about 70 and 80 kips can be achieved by driving the 12-inch diameter timber pile approximately 50 and 60 feet, respectively, below the CBP concrete slab.

The estimated penetrations to satisfy the required nominal (ultimate, unfactored) resistances can be determined from Figures 2 and 3 using appropriate resistance factors. For Strength Limit compression loading, we recommend a resistance factor of 0.65 for side and base resistance, in general accordance with American Association of State and Highway Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 4th Edition, with 2008 Interim Revisions (AASHTO, 2008). This value assumes that dynamic pile testing with signal matching is performed during installation of the production piles. In general accordance with AASHTO (2008), the required number of dynamic pile tests depends on the site variability. For the site variability and the proposed number of piles to support the CBP, we recommend that a minimum of two pile dynamic tests be performed. We understand that the dynamic pile testing will be performed during the installation of piles for the PCF.

The actual depth of pile penetration achieved will vary depending upon the consistency and relative density of the soil encountered during pile driving. The estimated penetrations into the dense sand and gravel and the driving resistance criteria may be modified after the initial production piles are driven.

We estimate that the steel closed-end pipe piles driven into the dense sand and gravel would experience settlements of about ½ to 1 inch under the proposed factored loads. These settlement estimates include the elastic compression of the pile as a result of the applied loading. For 50-foot timber piles bearing in soft deposits, we estimate settlements of about 2 to 4 inches.

TABLE 1
PILE DRIVING CRITERIA

Pile Type	Pile Diameter (inch)	Pile Wall Thickness (inch)	Estimated Pile Length (feet)	Nominal Compression Resistance (kips)	Continuous Pile Driving Blow Count (blows/foot)	Minimum Stroke (feet)	Maximum Compression Stress (ksi)
Steel pipe	18	3/8	122	200	12	6.0	26
Steel pipe	18	3/8	122	300	18	6.5	29
Timber	12	-	50	70	18	-	-
Timber	12	-	60	80	20	-	-

Notes:
1. Driving criteria for the steel pipe pile are based on GRLWEAP (Version 2005) analysis results.
2. Driving criteria for the steel pipe piles may be revised based on the results of the initial production pile installation.

Piles supporting the proposed temporary CBP will be installed through the existing fill and the underlying soft silt and sand deposits into the very dense sand and gravel deposits. Potential obstructions, such as wood and occasional concrete debris and very dense gravelly material may be encountered during the installation of the piles through the upper fill material encountered from the ground surface to about 10 to 15 feet bgs. Remedial measures such as predrilling and pre-excavation would be required to mitigate the impact of the potential obstructions. In addition to the above, the contractor may consider banding the pile tip and butt of the timber piles to reduce the potential for damage to the pile while driving through debris in the upper portion of the soil profile.

Sincerely,
SHANNON & WILSON, INC.

12/21/10

Robert A. Mitchell, P.E.
Associate

CIJ:RAM:JW/cij

Table 2 – Recommended Geotechnical Parameters for Development of L-Pile P-y Curves
Figure 1 – Site Plan
Figure 2 – Estimated Axial Resistance, 18-inch Diam., 3/8-inch Wall Thick, Closed-End Pipe Pile
Figure 3 – Estimated Axial Resistance, 12-inch diam., Timber Pile
Figure 4 – WEAP Analysis, 18-inch Diam., 3/8 inch Wall Thick, Closed-End Pipe Pile

REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 2008, AASHTO LRFD bridge design specifications: customary U.S. units (4th ed. with 2008 interim revisions): Washington, D.C., AASHTO, 1 v.

Landau Associates, 2009, Geotechnical data report, SR 520 pontoon construction design-build project, Aberdeen Log Yard, Aberdeen, WA, RFP Appendix G1, August 17.

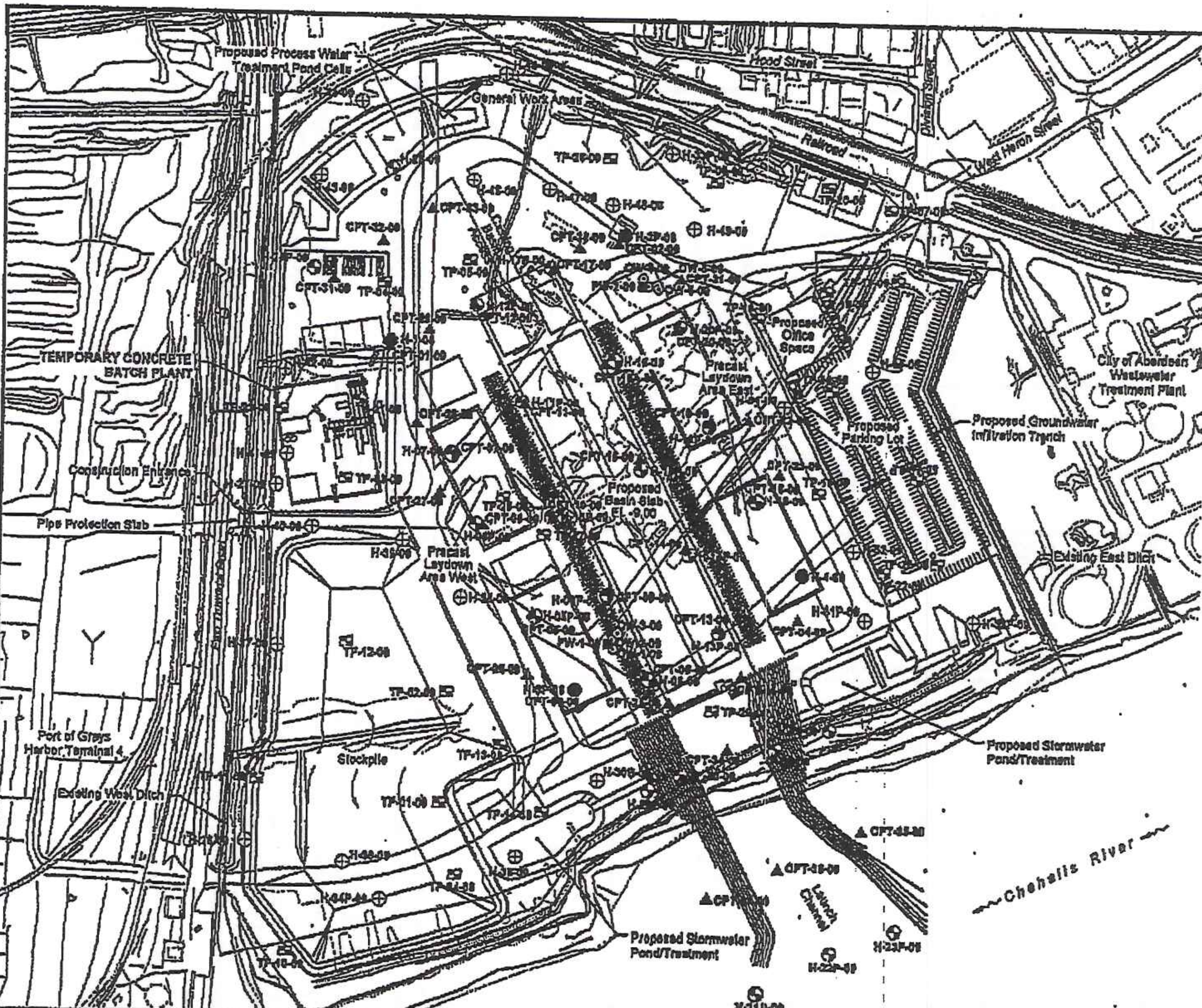
Pile Dynamics, Inc., 2005, GRLWEAP, one-dimensional wave equation analysis: Cleveland, Ohio, Pile Dynamics, Inc.

Shannon & Wilson, 2010, Geotechnical engineering recommendations, State Route (SR) 520 pontoon casting facility, Aberdeen, Washington, August 27.

TABLE 2
RECOMMENDED GEOTECHNICAL PARAMETERS FOR DEVELOPMENT OF L-PILE P-y CURVES

Top Elevation (feet)	Bottom Elevation (feet)	Soil Model	Total Unit Weight, γ (pcf)	Effective Unit Weight, γ' (pcf)	Average Cohesion, c (psf)	Friction Angle, φ (degrees)	Modulus of Subgrade Reaction, k	Static Softened Strain at 50% Max Stress, ε_{50} for Clay Model
					Static	Static	Static	
15	-11	Soft Clay	95	33	500	-	-	0.02
-11	-20	Reese Sand	120	58	-	32	50	-
-20	-35	Soft Clay	95	33	800	-	-	0.02
-35	-40	Stiff Clay w/o Free Water	100	38	1100	-	-	0.015
-40	-60	Reese Sand	120	58	-	30	35	-
-60	-75	Stiff Clay w/o Free Water	100	38	1100	-	-	0.015
-75	-105	Stiff Clay w/o Free Water	105	43	1500	-	-	0.01
-105	-121	Reese Sand	130	68	-	36	90	-
-121	-160	Reese Sand	135	73	-	38	125	-

Notes:
pcf = pounds per cubic foot
pci = pounds per cubic inch
psf = pounds per square foot
ksf = kips per square foot
ksi = kips per square inch



N

0 200 400

Scale in Feet

LEGEND

H-01-06 ● Borings by Others, Designation and Location, 2006

H-35-09 Borings by Others, Designation and Location, 2009 (Deep Borings - More than 50 Ft)

H-16-09 Borings by Others with Shearwave Logging and Pressuremeter Testing

H-34P-09 ⊕ Borings by Others, Location and Designation (Shallow Borings - Less than 50 Ft)

Denotes Piezometer Installation

CPT-01-09 ▲ Cone Penetration Tests by Others, Designation and Location

OW-1-09 Observation Wells by Others, Designation and Location

PW-1-09 Pumping Wells by Others, Designation and Location

TP-1-09 Test Pit Designation and Location

NOTE

This figure is based on *XL2572_AL_BP_Base Map-Default.dwg* and, *Figure 3-10.dwg*, received 5-10, and *49854_GEO_s_001.dgn*, received 7-28-10.

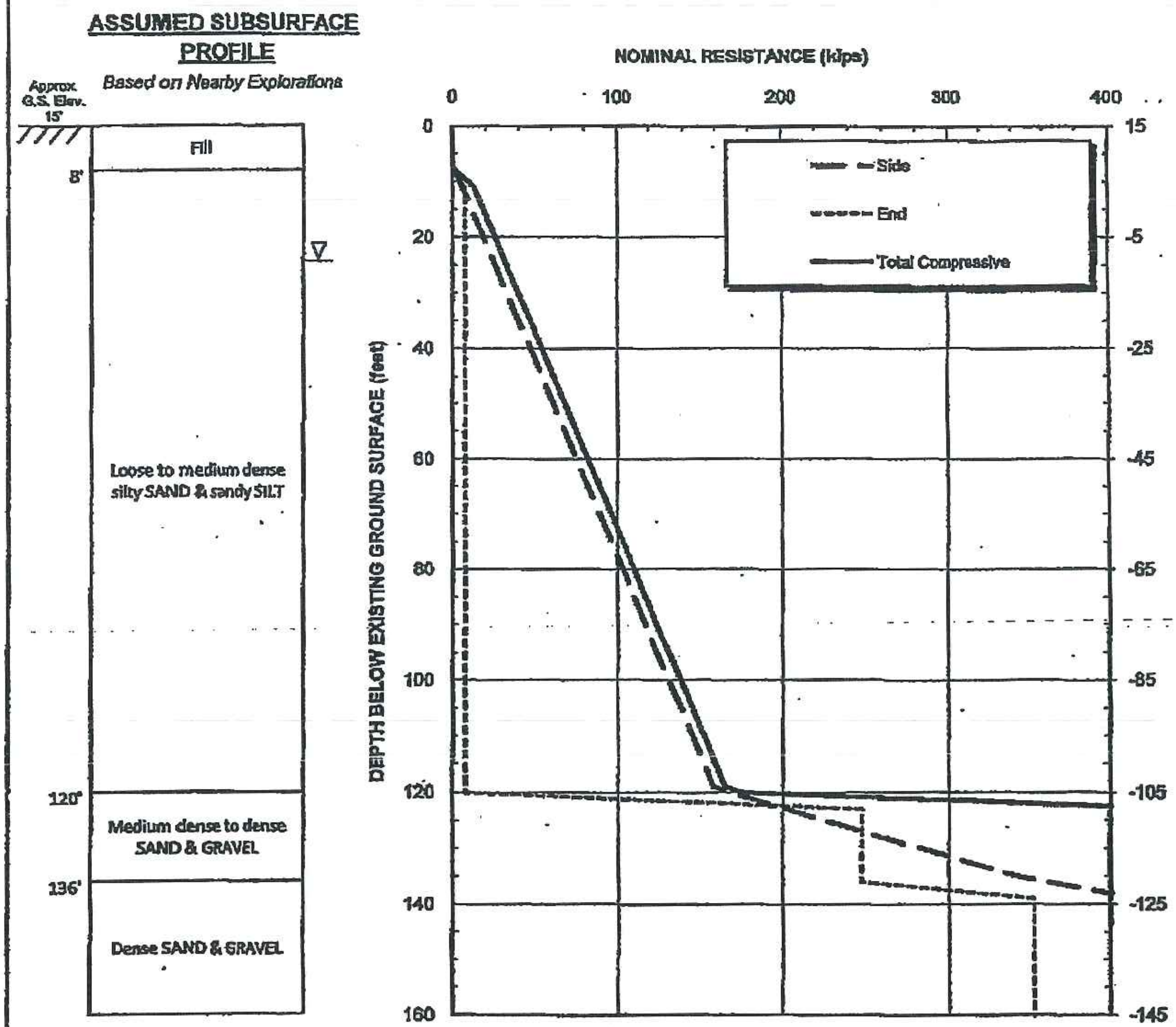
SR 520 Pontoon Casting Facility
Temporary Concrete Batch Plant
Aberdeen, Washington

SITE PLAN

December 2010 | 21-1-21453-001

SHANNON & WILSON, INC. | FIG. 1

39



NOTES:

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).

2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See note 3 for recommended resistance factors.

3. For the strength limit state, recommended resistance factors (RF) are:
   RF = 0.65 for both side and end resistance and
   RF = 0.35 for uplift resistance.
   Provided the appropriate number of dynamic testsing with PDA/CAPWAP are performed.

4. The downdrag load was not estimated for temporary piles.

SR 520 Pontoon Casting Facility
Temporary Concrete Batch Plant
Aberdeen, Washington

ESTIMATED AXIAL RESISTANCE
18-INCH DIA., 3/8-INCH WALL THICK
CLOSED-END PIPE PILE

December 2010 21-1-21453-001

SHANNON & WILSON, INC.
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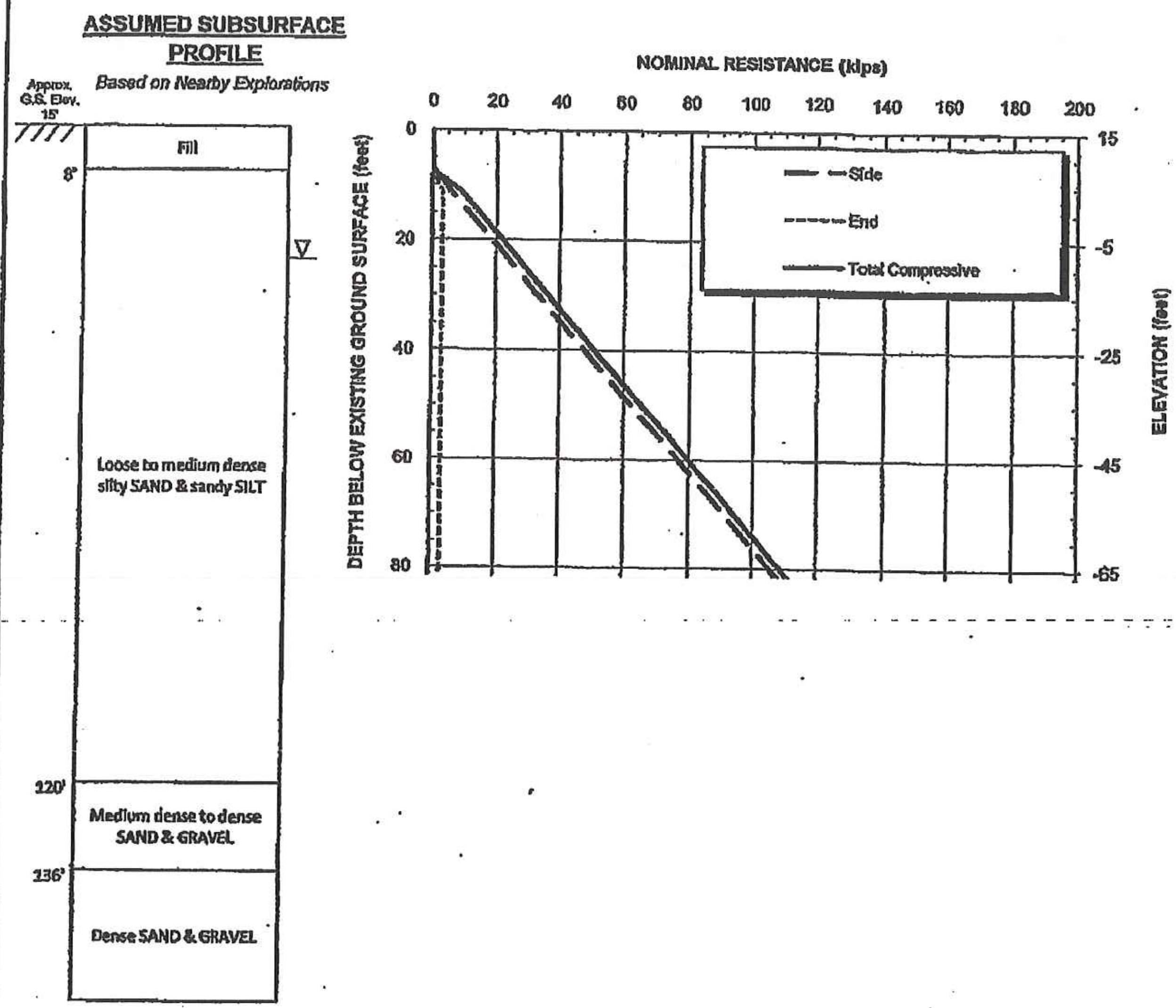
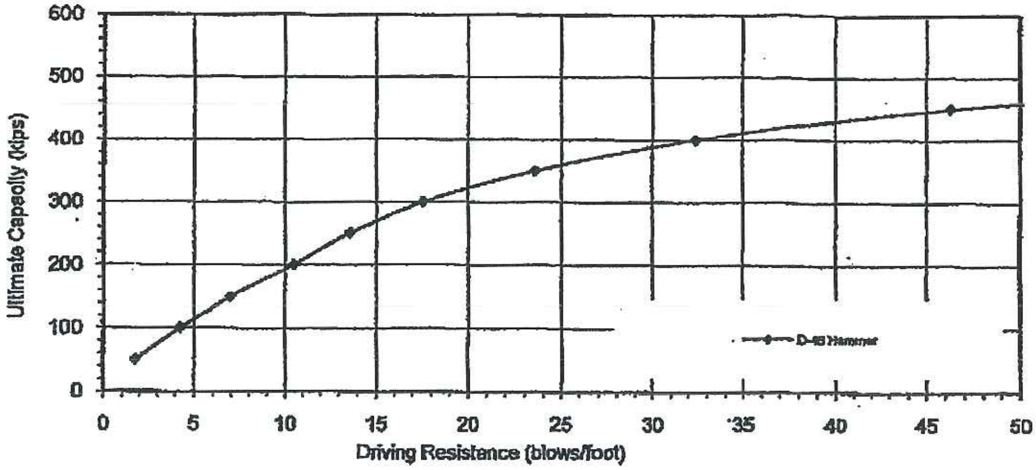
FIG. 2

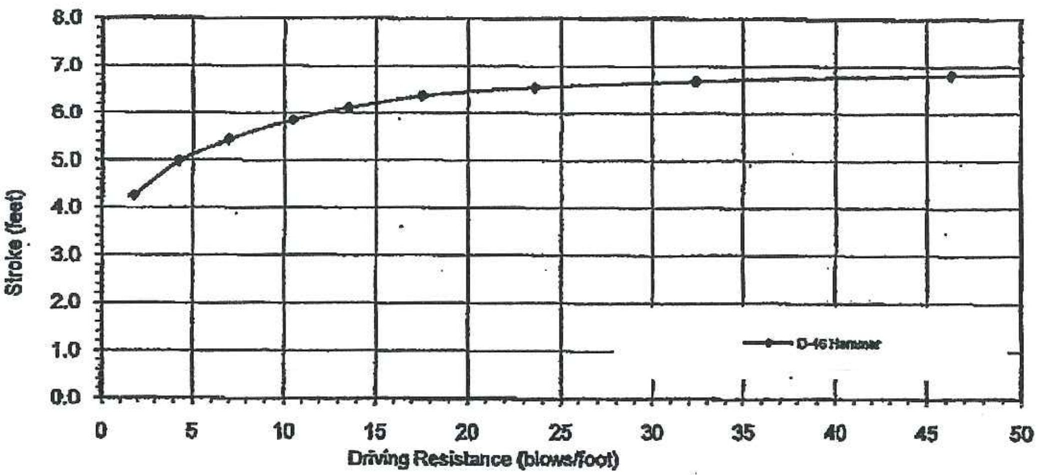


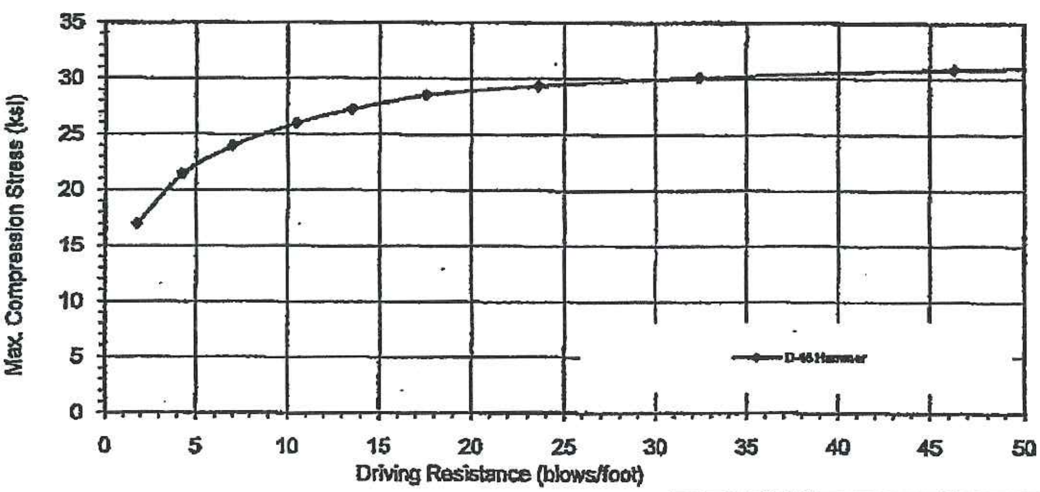
NOTES:

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See note 3 for recommended resistance factors.
3. For the strength limit state, see AASHTO LRFD (2008) for the recommended resistance factors.
4. The downdrag load was not estimated for temporary piles.

SR 520 Pontoon Casting Facility Temporary Concrete Batch Plant Aberdeen, Washington	
ESTIMATED AXIAL RESISTANCE 12-INCH DIA. TIMBER PILE	
December 2010	21-1-21453-001
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 3**







Notes:

1. The computer program GRLWEAP (PDI, 1998) was used for the WEAP analyses.
2. GRLWEAP recommended values used for quake, damping and helmet parameters.

SR 520 Pontoon Casting Facility
Temporary Concrete Batch Plant
Aberdeen, Washington

WEAP ANALYSIS
18-INCH-DIA., 3/8-INCH WALL THICK CLOSED-END PIPE PILE

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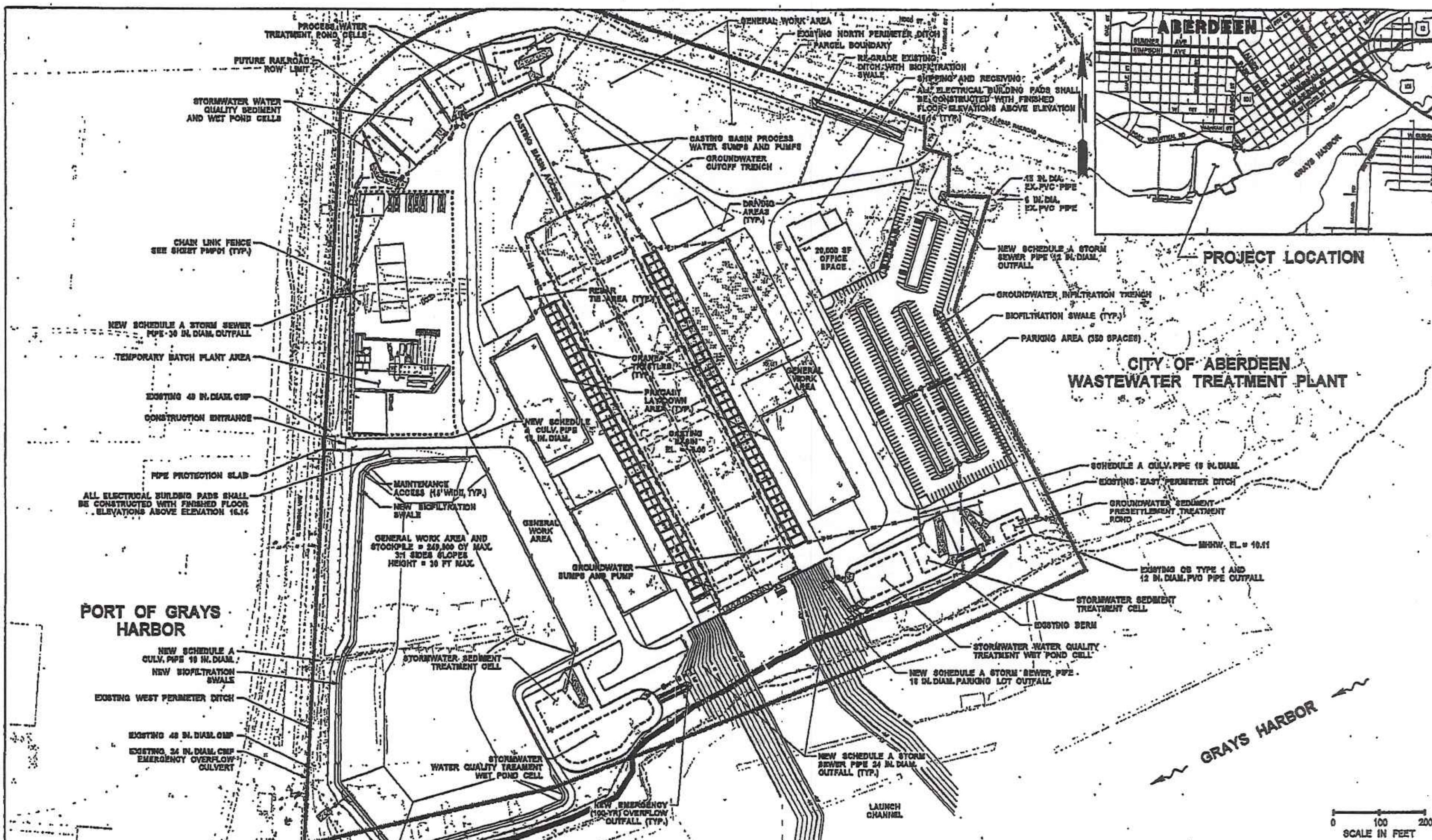
FIG. 4



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CHECKED BY	T. SCHNETZER			
PROJ. ENGR.	T. SCHNETZER	REV B - FINAL PLANS	01-14-11	RJK
REGIONAL ADM.	K. DAYTON	REVISION	DATE	BY

REGION NO.	STATE	FED.AID PROJ.NO.
10	WASH	
JOB NUMBER 007626		
CONTRACT NO. 007626		LOCATION NO.

Kiewit General HNTB kpff

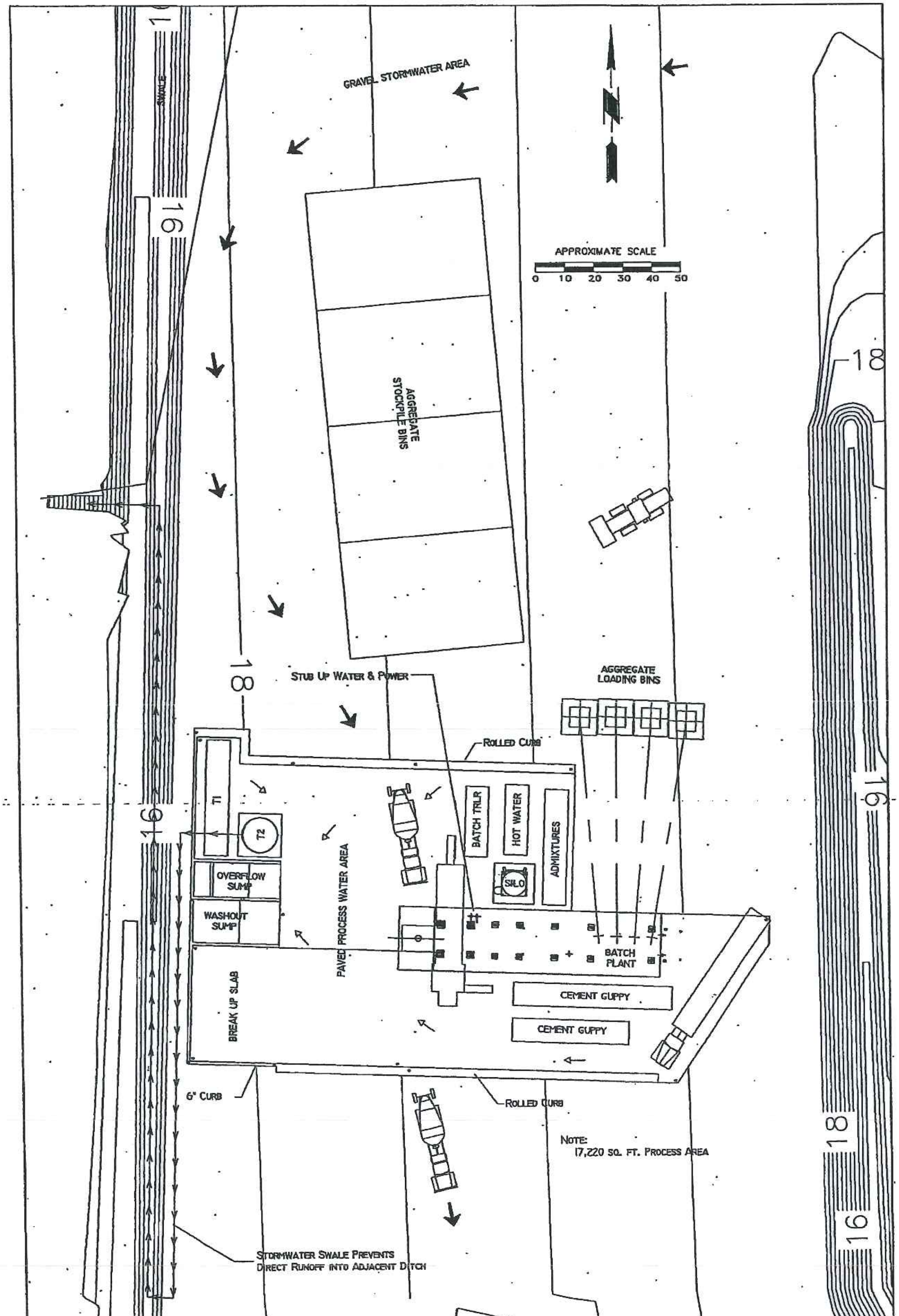
Washington State Department of Transportation

SR 520
PONTOON CONSTRUCTION
DESIGN-BUILD PROJECT
PACKAGE B - SITE FACILITY IMPROVEMENTS

SITE LAYOUT

GN03

SHEET 3 OF 257 SHEETS



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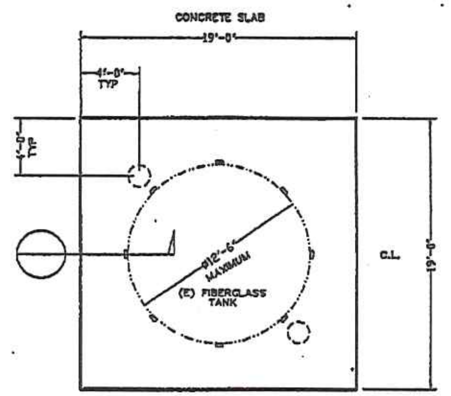

PLAN

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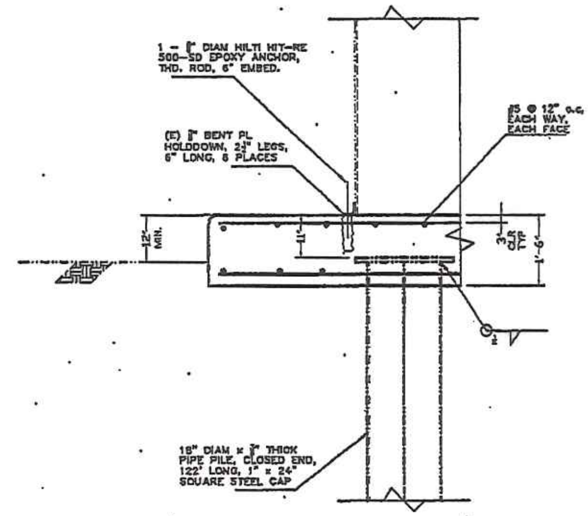

TYP. SECTION

NOTES:

1. f'c = 4,000 psi FOR CONCRETE
2. Fy = 60,000 psi FOR REINFORCING BARS
3. ADD SHALLOW, ROUNDED NOTCHES IN SLAB TO ACCOMMODATE DRAIN PIPES, ETC.
4. (E) MEANS "EXISTING"

VICTOR J. GHIGLERI
STATE OF WASHINGTON
37366
REGISTERED PROFESSIONAL ENGINEER
1/14/11
EXPIRES 06/26/11

PLEASE WORK SAFELY TODAY

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SCALE: NONE DATE: 01-11
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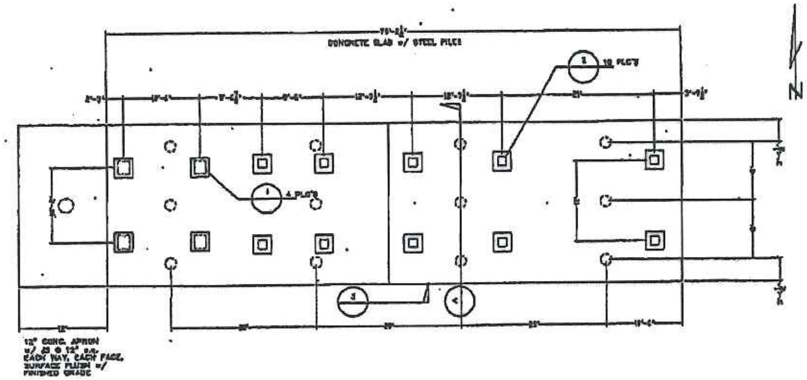
520 BATCH PLANT
TREATMENT TANK FTG.

CALIFORNIA PORTLAND CEMENT COMPANY

SHEET NO. 1 OF 1
DRAWING NO.
REV. 0



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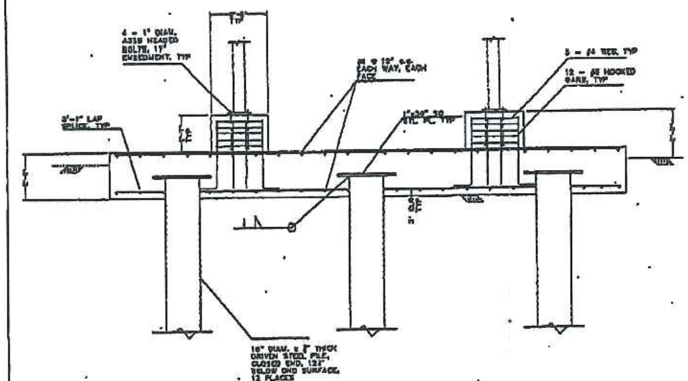

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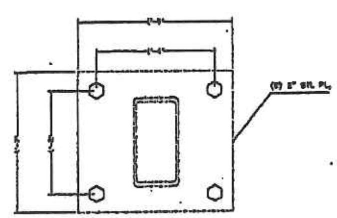

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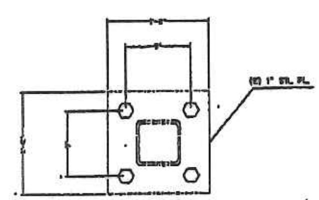

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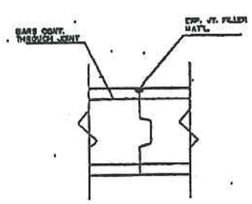

CONST. JT. DETAIL 3

VICTOR J. GINGLEN
STATE OF WASHINGTON
37366
REGISTERED
PROFESSIONAL ENGINEER

EXPIRES 06/26/11

NOTES:
1. CONCRETE: f'c = 4000 psi
2. REINFORCING BARS: ASTM A615 GR. 60
3. STEEL PLATE: A572

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PROJECT NUMBER:

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SR520 PONTOON CASTING
BATCH PLANT FOUNDATION

CALIFORNIA PORTLAND CEMENT COMPANY MOJAVE PLANT

SHEET NO. 1 of
DRAWING NO.
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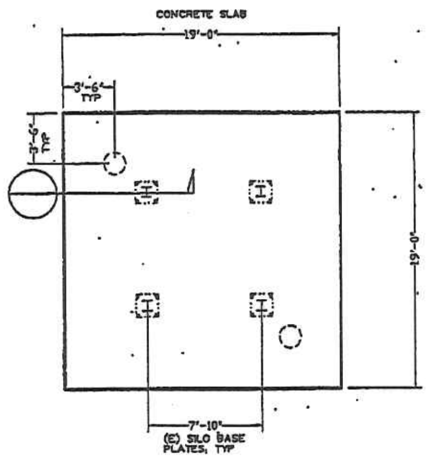
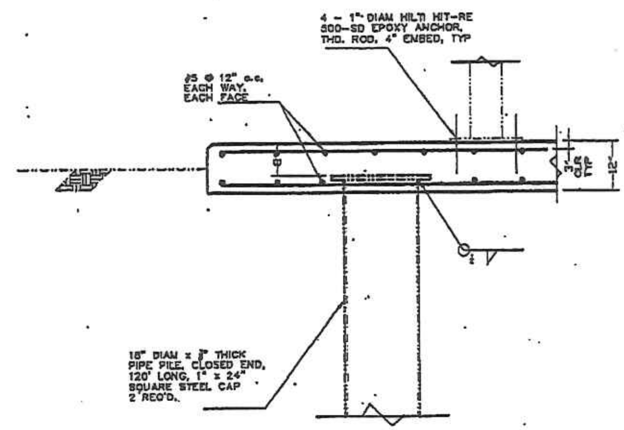


PLAN

MAXIMUM ALLOWABLE SILO
HEIGHT = 32 FEET



TYP. SECTION

NOTES:

1. f'c = 4,000 psi FOR CONCRETE
2. Fy = 60,000 psi FOR REINFORCING BARS
3. ADD SHALLOW, ROUNDED NOTCHES IN SLAB TO ACCOMMODATE DRAIN PIPES, ETC.
4. (E). MEANS "EXISTING"

VICTOR J. GUGLIELMI — STATE OF WASHINGTON — 37366 — REGISTERED PROFESSIONAL ENGINEER
1/14/11
EXPIRES 06/26/11

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520 BATCH PLANT
SILO FOOTING

CALIFORNIA PORTLAND CEMENT COMPANY

SHEET NO. 1 OF 1
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FILE: MY DOCUMENTS\PORT FACILITIES\SEATTLE\SR520 BRIDGE PLANT PILES\DWGS